

STEEL AND CONCRETE SUBSTRUCTURE
OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC

BY

J. A. HOLMBOE

ARMOUR INSTITUTE OF TECHNOLOGY

1914

624.6
H 73



**Illinois Institute
of Technology
UNIVERSITY LIBRARIES**

AT 342
Holmboe, J. A.
Design and estimate of cost
of steel superstructure and

For Use In Library Only

DESIGN AND ESTIMATE OF COST OF
STEEL SUPERSTRUCTURE AND CONCRETE SUBSTRUCTURE OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC.

A THESIS

Presented by

J. A. Holmboe

To The

PRESIDENT AND FACULTY

of

ARMOUR INSTITUTE OF TECHNOLOGY

For The Degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Having Completed The Prescribed

Course Of Study In

CIVIL ENGINEERING

1914.

Approved

Abel Philip
H M Raymond
Prof Civil Engineering

ILLINOIS INSTITUTE OF TECHNOLOGY
PAUL V. GALVIN LIBRARY
35 WEST 33RD STREET
CHICAGO, IL 60616

The Problem.

The problem consists of the design of a river crossing for highway traffic for a suburb of Chicago. The location and general conditions are assumed as shown on the blue print. The river is bridged by three spans, the middle span being seventy five feet and the two end spans each sixty feet. There will be two solid concrete piers and two reinforced concrete abutments.

List of Illustrations.

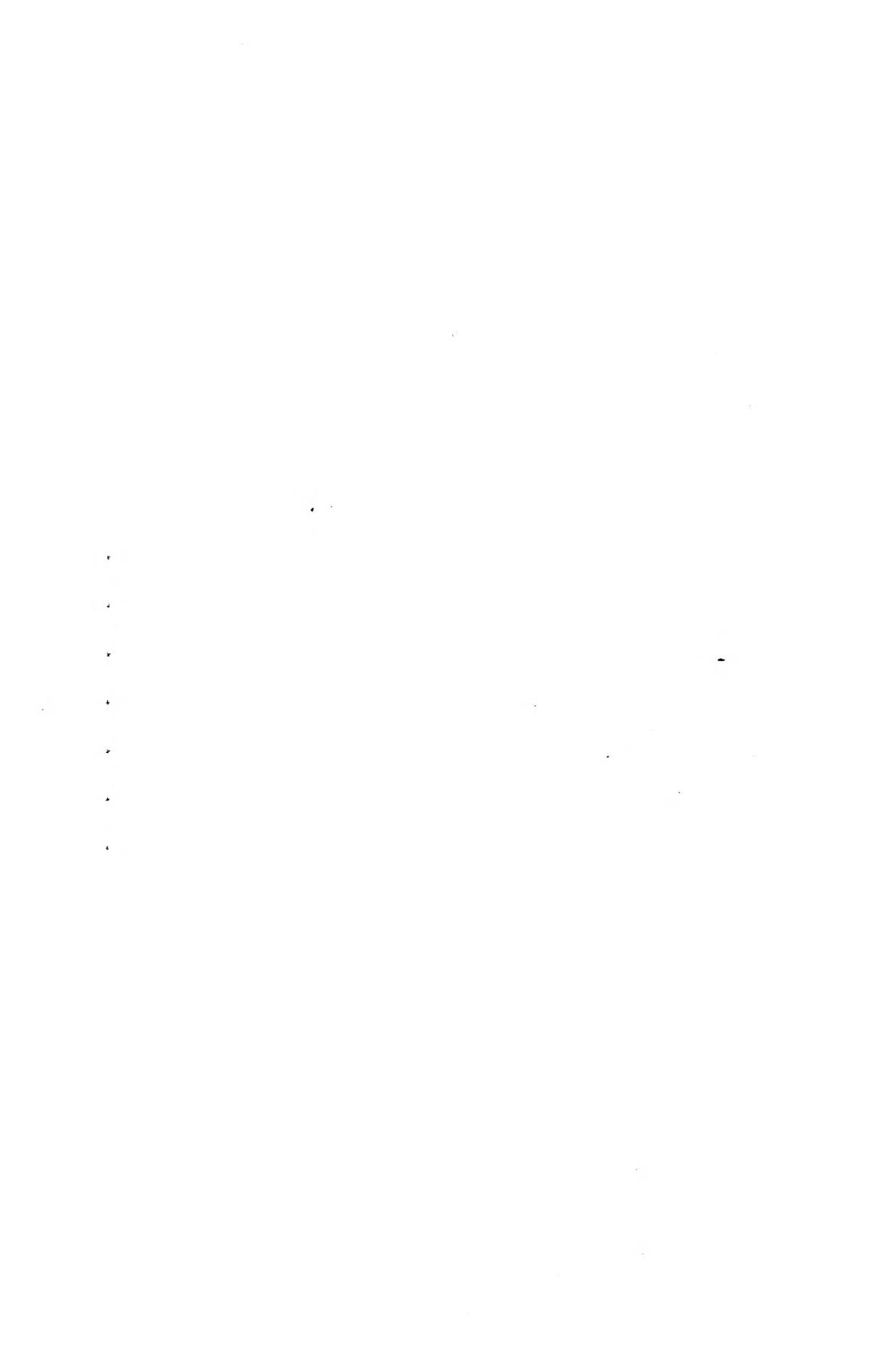
Design of 75' - 0" Pony Span.

General Plan of Abutments and Piers.

Design of Abutments and Piers.

Table of Contents.

	Page
Preface	2.
Design of Middle Span	3.
Design of End Span	24.
Design of Reinforced Concrete Abutment	33.
Design of Concrete Pier	44.
Estimate of Cost of Substructures	46.
Estimate of Cost of Superstructure	52.



Design of Middle Span.

Span = 75' - 0".

Weight = 3' - 0"

Width center to center of Trusses = 18' - 0".

Truss Trusses, parallel chords, no gusset plates.

Asphalt floor with concrete foundation.

Specifications - Illinois Highway Commission.

Class "C" bridge.

1/4" rivets.

Design of Floor.

Assume concrete foundation 4" thick. (FIG. 2)

Buckle plates = 5/16" thick.

Binder course = 1 1/2" thick.

Asphalt = 3" thick.

Total weight of floor taken as 160 # per cu.ft.

Weight of floor = 105 " per square foot.

Weight of floor per panel = 15 x 17 x 105 = 28725 "

Heavier wheel load = 4 tons = 8000 #.

Space the stringers 2' - 1 1/2" apart.

Design of Roadway Stringers.

To find the center of gravity:-

$$\frac{8000 \times 10}{16000} = 0.667 \quad (\text{fig. 3})$$



For the maximum bending moment, the heaviest load must be upon the span and a wheel load must be at the section and this wheel load causing the maximum bending moment must be as far on one side of the center as the center of gravity of all the loads is upon the other. In this case however the maximum bending moment is caused by a wheel load placed at the center of the span.

$\frac{10 - 5.667}{2} = 1.667'$ distance of center of stringer to center of Wheel " 1. This makes wheel " 2 come off the span. Then put wheel " 2 at the center of the span.

$$M = \frac{5000}{8} \times 7.5 \times 12 = 300000 "$$

$$\frac{300000}{13000} = 22.7 \text{ Section Modulus.}$$

Carnegie :- Try 10" I-beam @ 27.5 " $E = 33,3$

Weight of floor = 105 " per foot.

Weight of stringer = 27.5 " per foot.

Total weight = 130 " per foot. D.L.

$$\text{D.L.P.M.} = \frac{\pi r^2}{2} = \frac{130.5 \times 15^2}{2} \times 12 = 44800 "$$

$$\frac{44800}{13000} = 3.44$$

$27.7 + 3.44 = 31.14$ O.K. since 33.3 is greater than 31.14

Design of Track Stringers.

Live load is 16 tons on two axles 10 foot centers. (Fig. 4)

$$D.L. = \frac{0.12}{2} \times 7.5 \times 16 = 48000 \text{ "}$$

$$S = \frac{48000}{17000} = 31.8$$

Try 12" I-beam $\sigma = 31.8 \text{ "}$ $S = 32.0$

D.L. 103.0 "

31.5

40.0 T-beam

39.0 Fail

206.5 " Total.

$$S = \frac{w l^2}{g} = \frac{0.12}{2} \times \frac{12^2}{6} \times 16 = 9600 \text{ "}$$

$$69800 + 36100 = 431000$$

$$S = \frac{431000}{13000} = 33.16 \text{ " O.K.}$$

(Art. 57) $\frac{27}{2}$ is less than 33"

Design of Floor Joists.

Assume concentrated loads are supported by two stringers, but maximum dead load on one stringer



one unit at the end of the panel.

$$\text{Weight of the concrete floor} = 100 \times 15 \times 17.5'' \\ = 27350 \text{ lb}$$

$$8-12'' \text{ I-beams } 3 \times 15 \times 71.5 = 345 \text{ lb}$$

$$7-12'' \text{ T-beams } 7 \times 15 \times 37.5 = 630$$

$$\text{Rails } 3 \times 30 \times 47 = \frac{900}{5535}$$

$$\text{Add } 15'' \text{ for details} = \frac{868}{6366}$$

$$\text{Total weight} = \frac{6366}{6366}$$

$$\text{Assume weight of floor slab} = \frac{1400}{35000} \\ \text{Total Dead Load} = \frac{1400}{35000} = 39.77''$$

$$G.I.C. = \frac{1}{2} = 5000 \times 12.5 \times 12 = 30000 \text{ lb}$$

Live Load Computation.

There is a change for two kinds of traffic:-

(1) Street car and uniform load. Use this for moments. (Fig.5)

(2) Locomotive Engine and uniform load. Use this for shear. (Fig.6)

$$5000 + 5000 \times 2/16 = 5000 + 625 = 56250 \text{ lb} = 140000 \text{ lb}$$

Moment of street car.

Width 12' width = $10 \times 100 \times 5/16 = 334 \text{ ft-pf}$ per foot floor beam.

Condition of live load for maximum moment :-

$$\text{Reaction} = 2 \times 2500 = 5000$$

$$0.2 + 0.4 = 0.6$$

Our bonds are safe.¹⁶

100

$$\text{loned} = \text{lon} + \text{azimuth} = 180^\circ + \frac{120^\circ}{2} = 300^\circ$$

$$= 1944 \times 3 = 18000 \times \frac{1}{2} = 14000 = 7000 = 5600$$

$$= 25000 = 25000 \cdot 10^{-1} = 25000 \cdot 10^{-1} = 25000 \cdot 10^{-1} = 25000 \cdot 10^{-1}.$$

Total mon. t = 1200000 + 31700 = 12317000 "t" = 12317000
inc. bldg.,

Section 4.01 = $\frac{20000}{72} = 277.8$

100% 64" x 96" Poly T-Leaf = 100.0 sq. ft. per box

For a fair set of a nuclear engine against the
turbo-fan engine it is assumed to cover 10 x 10⁴
uniform load in before. (Fig. 6)

“Infection in the body is a disease of the blood.”

$$\text{Ex. } \frac{100}{x} + x^2 + 100x \text{ Ans. } x = 10$$

$$\text{left reaction} = \frac{25.1 \times 3}{15} + \frac{17.77}{15} + \frac{30.1}{15} = 24.57$$

$$+ \frac{16600}{100} \cdot 15 \cdot 70 = 3120 + 5000 + 77700 = 10670$$

100 types for 200 - long drive ~~120~~ ~~150~~ K-100

= 18000

Total connection weight = 2000 lb

Area Connection = 2000 / 2000 = 1 in. \times 1 in. \times 1 in.

Maximum Strainter Reaction.

Deadweight Stringers := (Fig. 7)

$$R = 8000 + 2/15 \times 1000 = 8533 \text{ lb}$$

$$3 \times 2 \times \frac{100}{15} \times 200 = 8000 \text{ lb}$$

$$D.W. = 100 \times 15 = 1500$$

$$\text{Total} = 8533 + 8000 + 1500 = 18033 \text{ lb}$$

Track Stringers.

$$R = 8000 + 3000 = 11000$$

$$\text{Dead Load} = 2000$$

$$\text{No uniform live load. } \underline{12000.5} \text{ Total}$$

Connection Angles on Stringers.

$\frac{3}{4}$ " rivets

$$\text{Act. S.S. } 17000 : 80/ = 8000 \text{ lb shear.}$$

$$8000 \text{ lb } 90^\circ = 10000 \text{ " bearing.}$$

Single shear = 7500

Double shear = 1750

Desiring Number of Rivets.

	3/16	5/8	7/16	1	5/16
8000	3750	4500	5000	6000	5500



For single stringers.

Lines AA and FF in bearing and double shear.

$$\frac{11493}{1780} = 1 \text{ rivet in web. (Fig. 10.)}$$

Rivets in floor beam are in single shear or bearing on web of floor beam.

$$\frac{11465}{3530} = 1 \text{ shop rivet in 3 floor rivets.}$$

Track Springs.

$$\text{Web} = 3/4"$$

$$\frac{12000.5}{1500} = 8 \text{ rivets}$$

$$\text{Web of floor beam} = \frac{10000.5}{4550} = 2 \text{ rivets or 2 islands.}$$

Use 10" I-beams.

Use C angles 4" x 4" x 1/2" x 1-1/2" Standard.

Floor No. 1 connection to post.

In web of floor beam the rivets are in double shear or bearing.

$$\frac{16850}{6000} = 7 \text{ rivets}$$

In post assume single shear, since post should not be thinner than 3/4".

$$\frac{16850}{3530} = 5 \text{ shop rivets or 5 floor rivets.}$$

Use C angles 4" x 4" x 1/2" x 1-1/2" Long. Cuts.



Load for Bridges.

1180 " per foot of car track.

75 " per foot of non-bridging superstructure.

Per panel per truck 75 ".

$$1180 \times 75 = 8850$$

$$75 \times 7\frac{1}{2} \times (17.5 - 15) = 1125$$

$$\text{Sum} = 8850 + 4170 = 13020 \text{ " L.L.}$$

Per panel per truck, Rail.

$$\text{Floor} = 17.5 \times 7\frac{1}{2} \times 100 = 13200$$

$$\text{Rails} = 50 \times 15 = 750$$

$$\text{Total} = 14650$$

$$\text{Steel in floor} = \frac{15}{3} \times 80 = 600 \text{ #}$$

$$\text{Roadway stringer} = 5\frac{1}{2} \times 35 \times 15 = 1845$$

$$\text{Track stringer} = 1 \times 71.5 \times 15 = 1075 \text{ "}$$

$$\text{Sum} = 600 + 1845 + 1075 = 3520 \text{ "}$$

10 lineal feet 6" x 4" x 7/8" angles at 12"

$$= 130 \text{ #}$$

$$130 + 3520 = 3650 \text{ say } 3700 \text{ #.}$$

This makes 1170 # per lineal foot of bridge per truck. Ceiling = 70" per lineal foot or 450 " per panel per truck.

Steel from Du Pont's Nominal

$$\frac{15}{16}(550 + 20 \times 5^2) = 562.5 \text{ per ft. per truss.}$$

$$562.5 + 100 = 662.5 \text{ per ft. per truss.}$$

$$17100 \times$$

Chord sec.

$$\tan \theta = \frac{8}{7.5} = 1.06666 \quad \theta = 46^\circ 30' \quad \sec \theta = 1.40173$$

$$P.D. = z = 17100 \times$$

Chord leg. = 7.5 m. at F.

$$\begin{aligned} \frac{8x}{11} \times \frac{17100}{11} &= 7.5(12 + \frac{1}{2})x = \frac{840x}{11} \\ &= \frac{3 \times 17100 \times 12}{11} = 36000 \times \end{aligned}$$

Chord SF. c. of m. at e.

$$\frac{8x}{11} \times \frac{17100}{11} = 7.5x = \frac{840x}{11} = \frac{3 \times 17100 \times 12}{11} = 36000 \times$$

Chord sec. = c. of m. at D.

$$\begin{aligned} \frac{8x}{11} \times \frac{17100}{11} &= \frac{17100}{11} \times 12 = \frac{20520}{11} \\ &= \frac{3 \times 17100 \times 12}{11} = 36000 \times \end{aligned}$$

Chord SF. c. of m. at c.

$$\frac{8x}{11} = \frac{3 \times 17100 \times 12}{11} = 36000 \times$$

Chord As. = c. of m. at b.

$$\frac{8x}{11} \times \frac{17100}{11} = \frac{17100}{11} \times 12 = 36000 \times$$



Diagonals.

Shear in panel FG = 0.

Therefore there is no dead load stress in the diagonals.

Shear in panel CE = $S_{ce} = \tau = w = 17100$.

$$\text{Stress} = 17100 \text{ tan}\theta = 25000 \text{ "} = \sigma_c = -50$$

Shear in panel AF = $I_w = 71200$.

$$\text{Stress} = 71200 \text{ tan}\theta = 50000 \text{ "} = \sigma_c = -100.$$

Footnote.

Foot Co.

$$\sigma_c \sin\theta + \sigma_o + \sigma_e \sin\theta = 0$$

$$-25000 \sin\theta + \sigma_o + 20000 \sin\theta = 0$$

$$\sigma_o = -(20000 - 25000 \sin\theta) \sin\theta = -10000 \text{ "}$$

Foot Re.

$$\sigma_c \sin\theta + \sigma_o + \sigma_e \sin\theta = 0$$

$$\sigma_o + \sigma_e = -25000 \sin\theta = -10000 \text{ "}$$

Live Load Stresses.

$$\text{Ratio} = \frac{10320}{17100} = 0.74$$

$$\sigma_d = \sigma_F = 71000$$

$$\sigma_e = 59400$$

$$\sigma_F = 47400$$

$$\Delta c = 23600$$

$$Ee = \frac{(1 + 2 + 3 + 4)}{5} w^1 \sec \theta = 10/5 \times 12100 \times 1.40173$$

$$= 36810 \text{ "}$$

$$AE = -Ee = -36810$$

$$De = \frac{(1 + 2 + 3)}{5} w^1 \sec \theta = 6/5 \times 12100 \times 1.40173$$

$$= 22100$$

$$Fe = -De = -22100$$

$$Fg = -Fe = \frac{(1 + 2 + 3)}{5} w^1 \sec \theta = 3/5 \times 12100 \times 1.40173$$

$$= 11050 \text{ "}$$

Eo.

$$Ee \sin \theta + Fe + Ee \sin \theta = 0$$

$$-368100 \sin \theta + -22100 + 70810 \sin \theta = 0$$

$$Co = -(368100 + 22100) \sin \theta = -17710 \times .70810$$

$$= -10720$$

Ee.

$$Fe \sin \theta + Ee + Fe \sin \theta = 0$$

$$-11050 \sin \theta + Ee + 22100 \sin \theta = 0$$

$$Ee = -(22100 - 11050) \sin \theta = -11050 \sin \theta = -8060 \text{ "}$$

Design of Upper Flange.

EF.

$$\text{P.L.} \quad 36800$$

$$\text{I.I.} \quad 21000$$

$$\text{Impact I.I.} \quad \frac{300}{\frac{300}{2} + 45} = \frac{528.00}{333.00}$$

$$C = 16000 - 70 \cdot 1/r$$

Assume section to be follows:-

$$1 \text{ cover tiles } 12'' \times 2'' = 6.00$$

$$2 \text{ angles } 6'' \times 4'' + 3/4'' = 12.12$$

$$27.77 \text{ sq. in.}$$

To find the center of gravity of this section.

take moment about the top.

$$y = \frac{6 \times 1.25 + 12.12 \times 3.75}{27.77} = \frac{7.5 + 45.21}{27.77} = 1.65 \text{ "}$$

Moments of inertia about the center of gravity.

$$I_x = 1/16 \times 12^3 \times (1.25)^3 + 6.00 \times (2.12)^2 = 17.68$$

$$I_y = 2 \times 31.1 + 12.12 \times (1.65)^2 = \frac{42.24}{21.61}$$

$$r = \sqrt{I/F} = \sqrt{\frac{42.24}{21.61}} = 1.87$$

$$C = 16000 - \frac{15.4 \times 1.87 \times 1.65}{1.87} = 16000 - 7401 = 14599$$

Take $C = 14599$ in calculating.

$$\frac{14599}{14000} = 103.5 \text{ sq. in.}$$

Private reqd. = $\frac{15.4 \times 14599}{16000} = 11 \text{ crop or 47 field}$

sq. m.

F.L. 64100

Z.L. 47400

Impacted $\frac{15.4}{14000} \times 64100 = 103.5$ Total

Use thickness of 1" and a 5000 psi.

Design of End Post.

A.T.

Use thickness of 1" and a 5000 psi.

$$\text{Length} = \sqrt{2^2 + 7.5^2} = \sqrt{100.0} = 10.0$$

D.L. 50000

L.L. 71820

$$\text{Impact} = \frac{21400}{12700}$$

$$C = 16000 - 20 \cdot 1/4 = 15800 - \frac{C_2}{1.00} \cdot \frac{1.00}{1.00} = 12300$$

$$= 12300$$

$$\text{Area req'd} = \frac{12300}{12700} = 0.96 \text{ sq. in. (Cast Iron)}$$

31 rivets required in upper chord.

Design of Lower Chord Tension member.

Lower Chord A.S.

D.L. 50000

L.L. 71820

$$\text{Impact} = \frac{21400}{300 + 750} = \frac{56800}{37300}$$

$$\text{Net Area req'd} = \frac{56800}{12700} = 11.0 \text{ sq. in.}$$

Try 2 angle 2" x 1" x 7/8" - Area = 11.0 = 11.00

Allow 2 rivet holes $\phi = .75"$ $\frac{1.5}{1.75 \times 2}$



Divide 20,000 by 7500 = 2.67 super rivets on 1' field.

$$\frac{11,400 \times 1000}{8000} = 141 \text{ super rivets on 1' field}$$

Tower Girder.

$$F.L. \quad 16000$$

$$L.L. \quad 10700$$

$$\text{Impact } \frac{350}{300 + 75} = \frac{4500}{10700}$$

$$\text{Net area } 1 - 1/4 = \frac{13500}{10700} = 12.5 \text{ sq. in.}$$

$$\text{Try C angles } 2" \times 4" \times \frac{3}{4}" = 17.85 \text{ sq. in.}$$

$$\text{Reduced G girder weight } .68 = \frac{1.05}{12.55} \text{ sq. in.}$$

$$\frac{11,400 \times 1000}{8000} = 141 \text{ super rivets on 1' field.}$$

Design of Post.

$$F.L. \quad 16000$$

$$L.L. \quad 10700$$

$$\text{Impact } 1.7. \quad \frac{350}{300 + 60} = \frac{5000}{8700} \text{ compression.}$$

$$C = 16000 - 50 \cdot 1/r$$

$$\text{Try C angles } 1" \times 5" \times 5/16" \quad A_{eq} = 7.19 \text{ sq. in.}$$

$$I = C \times 3.4 = 8.6$$

$$r = \sqrt{I/a} = \sqrt{\frac{8.6}{7.19}} = \sqrt{1.21} = 1.10$$

$$C = 16000 - \frac{50 \times 8.6 \times 1.10}{7.19} = 16000$$



$$\text{Area req'd} = \frac{276.5}{1000} = 0.2765 \text{ in.}^2 \text{ per in.}$$

$$\text{Rivets req'd} = \frac{1.14}{0.2765} = 4$$

Design of Diagonal.

Tensile Member.

$$\text{T.L.} \quad 30000$$

$$\text{L.L.} \quad 20010$$

$$\text{Impact L.L.} \quad \frac{300}{500+300} = \frac{750}{800} = 0.9375$$

$$\text{Net area req'd} = \frac{115410}{16000} = 7.21 \text{ in.}^2$$

Try S angles 4" x 3" x 11/16" - Gross area = 8.08

$$\text{Deduct S rivet holes @ .60} = \text{Net area} = \frac{7.48}{7.21}$$

$$\text{Rivets req'd} = \frac{7.48}{0.2765} = 27 \text{ rivets}$$

Tension Tension Member.

$$\text{T.L.} \quad 30000$$

$$\text{L.L.} \quad 23100$$

$$\text{Impact L.L.} \quad \frac{300}{500+300} = \frac{18450}{8000} = 2.30625$$

$$\text{Net area req'd} = \frac{25650}{16000} = 1.6 \text{ in.}^2$$

Try S angles 4" x 3" x 2" = Gross area = 8.08

$$\text{Deduct S rivet holes @ .60} = \frac{7.48}{7.08} = 1.04$$

Diagonal = Compute 100

Diamond plate = Tension.

$$F_c = -F_s = 65550$$

$$S = 10000 \rightarrow 80 \text{ l/r}$$

Try 3 angles 3" x 3" x 3" $\Delta = 10.12 \text{ in. i.e.}$

$$I = 3 \times 7.5 = 22.5$$

$$r = \sqrt{I/c} = \sqrt{\frac{22.5}{10.12}} = 1.63$$

$$S = 10000 = \frac{80 \times 11 \times 12}{1.63} = 720$$

$$\frac{65550}{720} = 8.16 \text{ sq. in. } \rightarrow \text{OK}$$

Divide by $\frac{10.12 \times 37.5}{8000} = 12 \text{ min. or } 1.5 \text{ sec.}$

Design of Diamond plate under Tensile.

F_c (compression) and F_u (tension) = 11000.

Live Load Stressing Only,

$$F_c.$$

$$I.L. \quad 11000$$

$$\text{Impact L.F. } \frac{600}{30 + 30} = \frac{1200}{60} = 200$$

$$S = 10000 = 80 \text{ l/r}$$

Try 3 angles 3" x 3" x 3" $\Delta = 8.16 \text{ in. i.e.}$

$$I = 3 \times 8.16 = 24.48$$

$$r = \sqrt{I/c} = \sqrt{\frac{24.48}{8.16}} = .605$$

$$S = 10000 = \frac{80 \times 11 \times 12}{.605} = 1200$$

$$\frac{1500}{1200} = 5.16 \text{ sq. in. required - ok.}$$

$$\text{Rivets required} = \frac{5.16}{.25} = 20.64 \approx 21 \text{ rivets}$$

Lateral Bracing.

(Fig. 8) According to specifications the wind load is 700 pounds per square foot dead load and 180 pounds per square foot live load. The panel dead load is $15 \times 300 = 4500$ lb.

$$\tan \theta = \frac{15.5}{15} = 1.033$$

$$\theta = 50^\circ + 30^\circ$$

$$\sec \theta = 1.78273$$

$$\text{L.L. per panel} = 15 \times 180 = 2700$$

Dead Load Stresses.

$$\text{Left reaction} = \frac{1}{2} \times 4500 = 2250$$

$$\text{Shear in panel EI for D.L.} = 0$$

Therefore no dead load stresses in diagonals.

Shear in panel EH.

$$\text{Shear} = (S_2 - \frac{1}{2} - 1)w = w = 4500$$

$$\text{Stress in Diagonals} = 4500 \sec \theta = 7100$$

Shear in panel AC.

$$\text{Shear} = (S_2 - \frac{1}{2})w = 2w = 9000$$

$$\text{Stress in diagonals} = 9000 \sec \theta = 14240$$

Liv. load up to e.

Panel FE. Live load up to e.

$$\text{Shear } \frac{12+3}{2} = 7.5W = 7.5 \times 2000 = 15000$$

Stress in diagonal = 15000 sec 60° = 2140 "

Panel CG. Live load up to e.

$$\text{Shear } \frac{12+3+3}{2} = 12W = 12 \times 2000 = 24000$$

Stress in diagonal = 24000 sec 60° = 4075

Panel AG. Live load up to e.

$$\text{Shear } \frac{12+3+3+3}{2} = 18W = 18 \times 2000 = 36000$$

Stress in diagonal = 36000 sec 60° = 7100

Total stress in Eg = 2140

$$\text{Area req'd} = \frac{2140}{18000} = .114$$

Try one angle $\frac{3}{2}" \times 3" \times 3/8"$ Area = 0.3 sq.in.

Net area = 0.3 - .114 = 0.186 sq.in.

$$\text{Shop rivets req'd} = \frac{1.35 \times 10}{\pi} = 4 \quad \text{or } 6 \text{ field.}$$

T.L. 42000

C.L. $\frac{7100}{11345}$

$$\text{Area req'd} \frac{11345}{10000} = .11345 \text{ sq.in.}$$

Try 1 angle $\frac{3}{2}" \times 3" \times 3/8"$ Area = 0.75 sq.in.



$$\text{Net area} = 0.76 + .38 = 1.14 \text{ sq ft.}$$

1 crop river or 2 fields.

Area

$$D.L. \quad 14.40$$

$$D.L. \quad \frac{\text{size}}{\text{size}} \text{ total}$$

$$\text{Area req'd. } \frac{\text{size}}{1600} = 1.57 \text{ sq. in.}$$

$$\text{try 1 angle } 72^\circ \times 7" \times 7/8" \quad \text{Area} = 0.76 + 1.14.$$

$$\text{Net area} = 0.76 + .38 = 1.14 \text{ sq ft.}$$

1 crop river or 2 fields.

Design of Deck.

The dimensions of this bridge are the same as for the middle span except that the length is sixty feet instead of seventy-five feet. There will be four panels of fifteen feet and the height will be eight feet as was the case in the preceding design.

Design of Floor.

The design of the floor will be the same in this case as in the preceding design.

Design of Stringers.

Since one panel length is the same for both bridges, the design of the stringers will be the same.

Design of Floorbeams.

The floorbeams will be the same for both bridges. The connection angles of the stringers and floorbeams will also be the same.

Load for Trusses.

The weight of the floor per panel will be the same or 13300."

Steel from Du Four's formula -

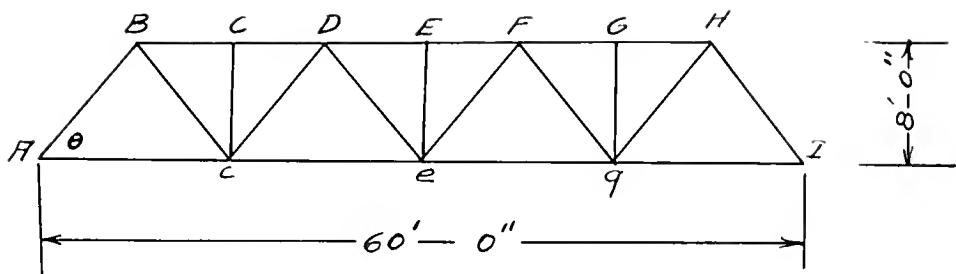
$$\frac{15}{6} (250 + 2.5 \times 60) = 3000 "$$



$$10800 + 3000 = 10500 \text{ " per panel per true L.L.}$$

The live load per panel per truss will be the same in both cases = 10500 "

Processes.



$$\theta = 40^\circ - 30^\circ = 10^\circ \quad \sec \theta = 1.10153$$

$$L.L. = w = 10500 \text{ "}$$

Chord DE, c. of m. at e.

$$\frac{1}{2}w \times 2x = \frac{1}{2}w = \frac{1}{2} \times 10500 \times \frac{15}{8} = 63750$$

Chord ce, c. of m. ab D.

$$\frac{1}{2}w \times \frac{15}{8} = \frac{1}{2}w = \frac{1}{2} \times 10500 \times \frac{15}{8} = 63750$$

$$= 1.5 \times 63750 \times 15 = 57100$$

Chord ED, c. of m. at e.

$$\frac{1}{2}w \times x = \frac{1}{2}w = \frac{1}{2}w \times 10500 \times \frac{15}{8} = 63750 \text{ "}$$

Chord Ac, c. of m. at D.

$$\frac{1}{2}w \times \frac{15}{8} = \frac{1}{2}w = \frac{1}{2} \times 10500 \times \frac{15}{8} = 63750 \text{ "}$$

F1 60deg.

$$\text{Shear in panel } D = \frac{1}{2} \sigma - \tau = \sigma - \frac{1}{2} \tau = -\frac{1}{2} \tau$$

$$\text{Stress} = 8400 \text{ sec } \theta = 16200 \text{ sec } 60^\circ = -7200$$

$$\text{Shear in panel } C = \frac{1}{2} \sigma + \tau = \sigma + \frac{1}{2} \tau = 8400$$

$$\text{Stress} = 8400 \text{ sec } \theta = 16200 \text{ sec } 60^\circ = 7200$$

$$\text{Shear in panel } A = \frac{1}{2} \tau = 3600$$

$$\text{Stress} = 8400 \text{ sec } \theta = 16200 \text{ sec } 60^\circ = 7200$$

Eq. 3.10.10.

First Eq.,

$$De \sin \theta + Ce + De \sin \theta = 0$$

$$-12700 \sin 60^\circ + 3600 + 7200 \sin 60^\circ = 0$$

$$Ce = -(3600 + 12700) \sin 60^\circ = -26700 \sin 60^\circ = -17150$$

First Eq.,

$$De \sin \theta + Ce + De \sin \theta = 0$$

$$16200 \sin 60^\circ + Ce + 10000 \sin 60^\circ = 0$$

$$Ce = -3 + 16200 \sin 60^\circ = -10000$$

Linearize differences,

$$\text{Ratio} = \frac{16200}{10000} = .75$$

$$De = 12700$$

$$ce = 12250$$

$$CD = 36400$$

$$Ac = 17700$$



$$P_c = \frac{(1+\beta)}{4} \pi r^2 N^2 \quad \text{sec } \theta = \beta / 1 \times 10000 \times 1.40173 = 1.40173$$

$$= 27600$$

$$AB = -P_c = -27600$$

$$Ze = \frac{(1+\beta)}{4} \pi r^2 \sec \theta = \frac{3}{4} \times 10000 \times 1.40173 = 17800$$

$$De = -Ze = -17800$$

$$22^\circ$$

$$P_c \sin \theta + Ze + De \sin \theta = 0$$

$$-17800 \sin \theta + Ze + 27600 \sin \theta = 0$$

$$Ze = -(27600 - 17800) \sin \theta = 10000 \text{ "}$$

Design of Upper Chord.

P.L.	23000
------	-------

L.L.	17200
------	-------

Impact I.L.	$\frac{300}{300+30} \times 23000 = \frac{20000}{15000}$
-------------	---

$$S = 18000 + 70 \cdot 1/4$$

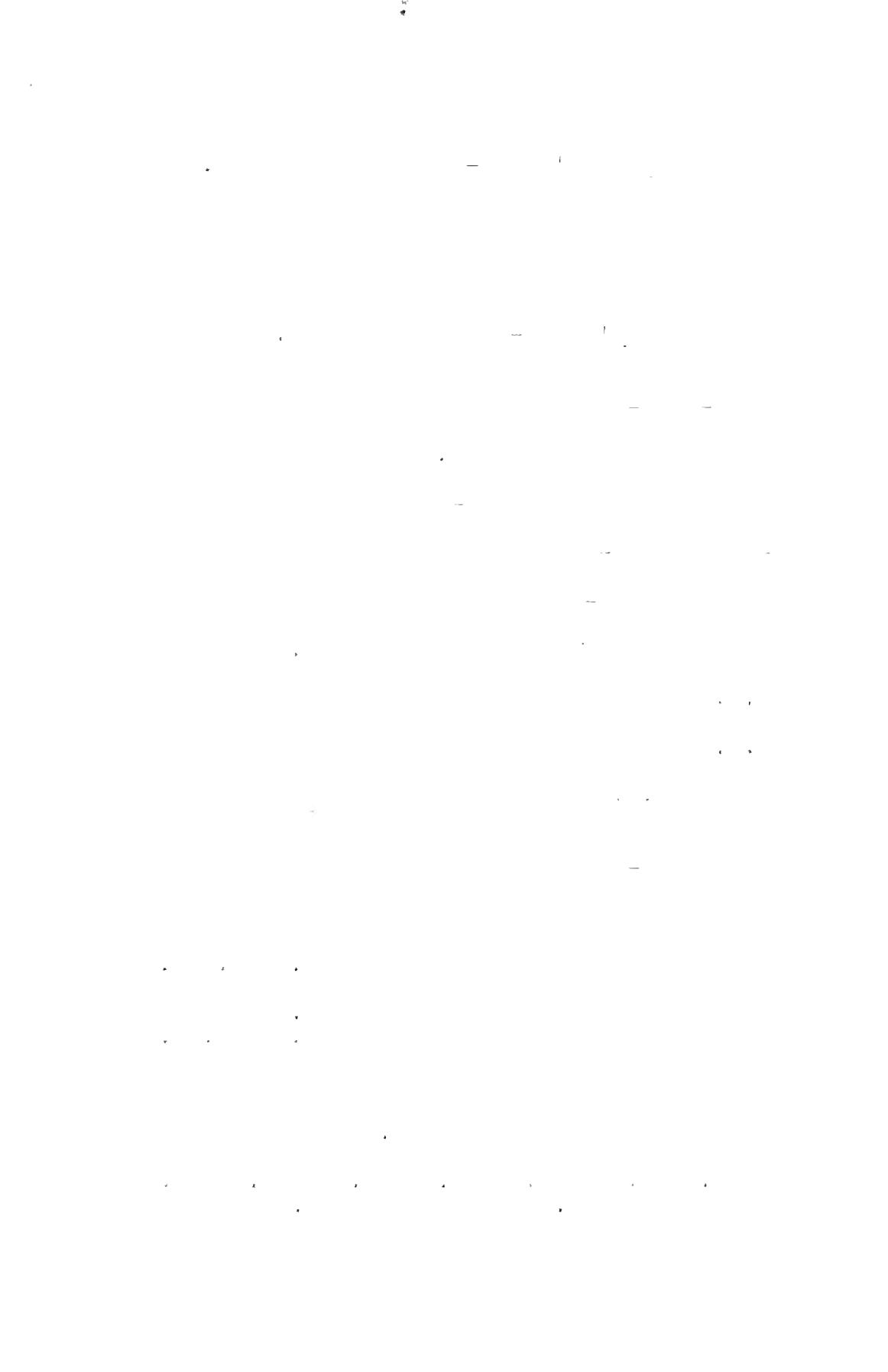
Assume section as follows:-

$$1 \text{ Cover plate } 12'' \times 5/8'' = 1.5 \times 1 \times 12.$$

$$2 \text{ angles } 3'' \times 1'' \times 1/2'' = \frac{0.5 \times 1}{12 \cdot 08} \text{ sq.in.}$$

To find the centre of gravity of the section
take moment about the top.

$$Y = \frac{1.5 \times 12.7 + 2.5 \times 5.315}{14.00} = \frac{9.45 + 33.275}{14.00} = 1.675$$



Moments of inertia about the center of gravity:-

$$I_x = 2/12 \times 18 \times (.675)^3 + 1.5 \times (1.75)^2 = 10.0.$$

$$I_y = 2 \times 18.1 + 9.00 \times (.315)^2 = \underline{6.66}$$

$$= 2.16$$

$$I = \sqrt{I_x/I_y} = \sqrt{\frac{10.0}{6.66}} = \sqrt{1.51} = 1.23$$

$$C = 18000 = \frac{10.7}{1.23} \frac{1.5}{1.23} \times 12$$

$$= 18000 = 1160 = 17380$$

$$\frac{153100}{15880} = 11.07 \text{ sec. in a reg'tl. 30 ft.}$$

Pivots req'd = $\frac{14.12}{8000} \times 13800 = 23$ snap or 70 field.

	Sec's.
I.L.	17280
Impact	<u>30800</u>
	24340 Total

Use same section as for 27.

Design of Rail Tote A.

$$\text{Target} = 111 - 0"$$

Use same section as for 27.

	Sec's.
I.L.	30800
Impact	<u>32300</u> 37400

$$C = 16000 - 80 \cdot 1/r, r = 10000 = 2000 \frac{1}{10000}$$

$$= 16000 - 8000 = 1^{\circ} 100$$

$$\text{Area req'd} = \frac{77410}{1100} = 7.037 \text{ in}^2 \text{ or } 0.47 \text{ in. } \times 1.6 \text{ in.}$$

SS shop rivets - 35 sheet.

Design of Lower chord Tension members.

sc

$$\text{P.L.} \quad 55100$$

$$\text{L.L.} \quad 41300$$

$$\text{Impact} \quad \frac{700}{300+70} \quad \frac{33000}{129600}$$

$$\text{Net area req'd} = \frac{169250}{13000} = 0.13$$

$$\text{Try 2 angles } 6" \times 4" \times \frac{1}{8} = \quad 0.10$$

$$\text{deduct 2 rivet holes} \times .11 = \quad -.022$$

0.078 in. \times 1.00

$$\frac{1.00 \times 10000}{6000} = 16 \text{ in. } \times \text{ rivets or } 27 \text{ in. } \times \text{ rivets}$$

Use the same section for the whole lower chord.

Design of Tors.

sc.

$$\text{P.L.} \quad 47050$$

$$\text{L.L.} \quad 10000$$

$$\text{Impact L.L.} \quad \frac{700}{300+70} = \frac{0.200}{0.270} \text{ Comp.}$$

$$C = 16000 - 80 \cdot 1/r$$

$$\text{Try 2 angles } 4" \times 3" \times 5/16" \quad \text{Area} = 1.19 \text{ in. } \times \text{in.}$$



$$I = 2 \times 7.4 = 14.8$$

$$z = \sqrt{I/a} = \sqrt{\frac{14.8}{1.0}} = 3.86$$

$$S = 10000 = \frac{20 \times 3.86 \times 10}{1.0} = 10000$$

$$\text{Area req'd} = \frac{10000}{3.86} = 2.63 \text{ sq. in.} \rightarrow \text{sect. area}$$

Design of Rectangle.

Poly-Tension member.

$$\text{D.L.} \quad 30000$$

$$\text{L.L.} \quad 20000$$

$$\text{Impact} \quad \frac{30000}{300 + 45} = \frac{80000}{345} = 227.3$$

$$\text{Net area req'd} = \frac{80000}{345} = 2.27 \text{ sq. in.}$$

$$\text{Try 2 angles } 4" \times 3" \times 3/16" \quad \text{true area} = 2.04$$

$$\text{deduct 2 rivet holes} \times .45 = \frac{2.04 - 0.90}{2} = 0.57$$

Poly-Compression member.

$$\text{D.L.} \quad 12000$$

$$\text{L.L.} \quad 13000$$

$$\text{Impact} \quad \frac{30000}{300 + 45} = \frac{100000}{345} = 290.5$$

$$A = 10000 - 45 \cdot 1/2 =$$

$$\text{Try 2 angles } 4" \times 7" \times 2" \quad A = 6.70$$

$$I = 2 \times 7 \cdot 0 = 14.0$$

$$z = \sqrt{I/b} = \sqrt{\frac{14.0}{6.70}} = \sqrt{2.07} = 1.44$$

$$C = 16000 - \frac{80 \times 11 \times 12}{1.34} = 16000 - 8560 = 7440$$

$$\frac{7440}{7480} = 5.03 \text{ min. req'd. O.R.}$$

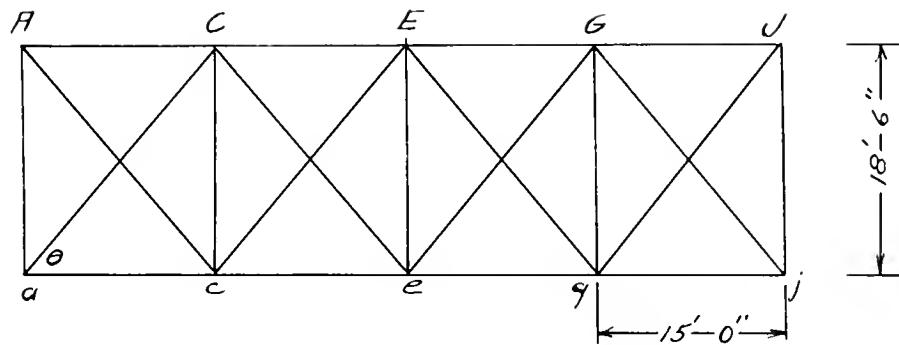
Diverts req'd. = $\frac{2.5 \times 7480}{9600} = 1.5$ acre or 11 field

Lateral Bracing.

P.L. per panel = 4500 same as in previous design.

I.L. per panel = 2000 " " " " " "

Dual Load Stresses.



$$\sec \theta = 1.58736$$

$$\text{Shear in panel } CE = (\frac{1}{2} + \frac{1}{2} - 1)w = \frac{1}{2}w = 2500$$

$$\text{Stress in diagonals} = 2500 \sec \theta = 3500$$

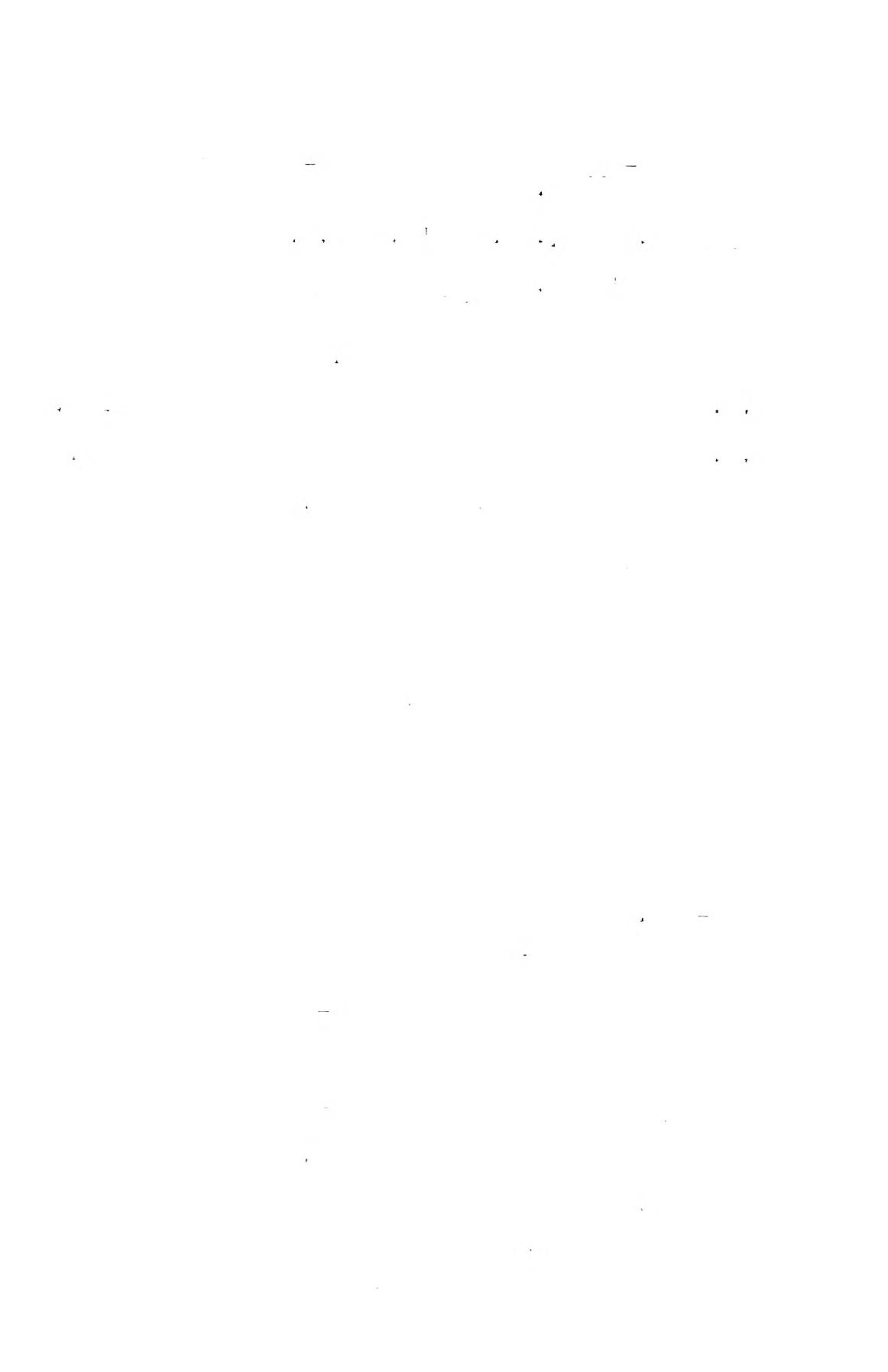
$$\text{Shear in panel } AC = (\frac{1}{2} + \frac{1}{2})w = 1.2w = 3750$$

$$\text{Stress in diagonals} = 3750 \sec \theta = 50700$$

Live Load Stresses.

Panel CE. Live load up to e

$$\text{Stress in diagonals} = 1570 \sec \theta = 20000$$



Panel AC. First decide area.

$$\text{Area} = \frac{1+2+2}{4} \times 2/4 = 2.250 = 2.25\text{ in}^2$$

Stress in Web example = 57100. sec = 5710

Stress in Ac.

$$\text{D.F.} \quad \frac{2.250}{10000}$$

$$\text{L.I.} \quad \frac{5710}{10000}$$

$$\text{Area req'd} = \frac{12100}{10000} = .121$$

Try 1 angle $\frac{\pi}{2}'' \times 1'' \times 3/8''$ Area = $\frac{\pi}{2} \times 1 \times .375$

$$\text{Net area} = \frac{\pi}{2} - .75 = 1.26 \text{ sq.in.}$$

Two rivets = $\frac{1.26 \times .25}{2} = 1$ or 0.5 in dia rivet.

Design of Reinforced Concrete Abutments.

Design footings or piling for design hydraulic, footings to take all loading. Flex footings to be set on piles.

$$\text{Piling} = \text{Dredge load per ft} = \frac{\text{C.S.}}{2} + \frac{\text{W.}}$$

$$= \text{Capacity of pile} = 1000"$$

$$n = \text{Allowable load per sq. in.} = 201 - 0",$$

$$r = \text{penetration of pile for load } R = 1".$$

Weights and loadings:-

$$\text{Ground line} = 150" \text{ above bridge footer.}$$

Weight of bridge determined from design of a 75' Highway Bridge - Class C.

Weight of live load - Superelevation included.

Teaching Abutment :-

Courtesy gravel.

1 tonne per square feet allowable.

"C.S.".

$$\text{Weight of bridge} = 200000",$$

200000 = 100000" carried by abutment,

= 5000 " carried by each bridge footer.

Plusses = 101 - 0" center to center.

Flex span = 201 - 0".



Plane traction figure with no load - per
load directly over a abutment end is one-half
of the bridge. Then calculate the tractive force
 $\frac{7}{16}$ of the weight of the normal wheel load
plus $\frac{3}{4}$ of the weight of the lighter wheel
load.

$$\text{Heavier wheel} = 8,700 \text{ lb} = 16000 \text{ lb}$$

$$\text{Lighter wheel} = 4,500 \text{ lb} = 8000 \text{ lb}$$

$\frac{7}{16}(16000 + 8000) \times 8000 = 12000 \text{ lb}$ for each trile
seat. Distribute load over $12\text{ ft} = 6\text{ in.}$

$$\frac{12000}{18} = 1417 \text{ lb per linear foot load.}$$

$$\frac{51200}{20} = 4071 \text{ lb per linear foot load.}$$

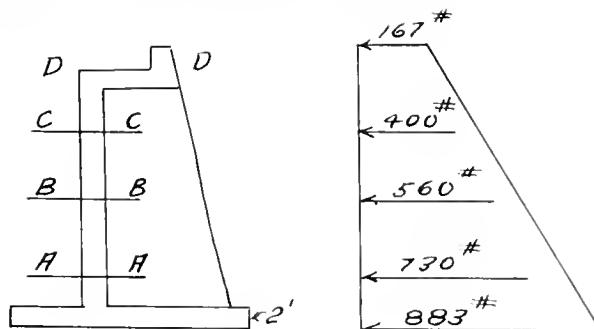
T.L. of 100" per square foot of running floor
space given by $130 \times 14.5 + 15 \times \frac{16000 + 8000}{18}$

$$= 30 + 150 = 180 \text{ " per linear foot.}$$

$$\text{T.L.} = 4071 \text{ " per linear foot.}$$

$$\text{T.L.} = 1417 + 730 + 180 = 2327 \text{ " per linear foot.}$$

$$\text{Total} = 4071 + 2327 = 6400 \text{ "}$$





$$\text{Forecharge} = \frac{\pi C_0 \rho_0 A_0}{2} = 100 \text{ kN/m}^2$$

Counterforce =

$$\text{Measure at top} = 250 \text{ kN}$$

$$\text{Measure at base} = 200 + 250 = 450 \text{ kN}$$

$$\text{Resultant} = \sqrt{250^2 + 450^2} = 511.5 \text{ kN}$$

Design of section A - A.

Unit pressure at A - A = $\frac{450}{100} = 4.5 \text{ kN/m}^2$ (approx)

Design as a simple beam section.

$$x = 1/12 \times l^2 = 1/12 \times 750 \times 3^2 = 56.25 \text{ mm} = 56.25 \text{ mm}$$

$$\delta = \sqrt{\frac{M}{E}} = \sqrt{\frac{450 \times 3}{200000}} = \sqrt{39.7} = 6.3 \text{ mm}$$

Use a uniform section of 100 mm.

$$I_s = 150000 \text{ mm}^4 \quad r_y = 25 \text{ mm} \quad I = 5/3 \text{ mm} \quad D = 1 \text{ mm}$$

$$\text{For section } 100 \times 100 = \frac{250}{100} = 250 \text{ kN}$$

$$I = \frac{\pi}{64}$$

$$I = \frac{\pi}{64} = \frac{200}{100 \times 100} = 0.00200 \text{ mm}^4$$

$$\text{Steel area, } A = \frac{A}{\pi} = \frac{100 \times 100}{100 \times 100 \times 3.14 \times 10^{-6}} = 0.796 \text{ mm}^2$$

"0.2" is total cross-section "area" of A = $0.2 \times 100 \text{ mm}^2$.

Section P = 5.

SI = 0" above the base.

Unit pressure = 520 lb./sq.in. (calculated)

$$A = 1/12 \times 700 \times 24 \times 12 = 70043 \text{ in}^2$$

$$\text{Steel area} = A = \frac{\pi d^2}{4} = \frac{\pi 12^2}{4} = 113.09 \text{ in}^2 = 1.13 \text{ in}^2$$

The $\frac{1}{2}$ " ϕ 2024 spud is the strongest. Area = .625 sq.in.

Section P = 6.

SI = 0" above the base.

Unit pressure = 480 lb./sq.in. (calculated)

$$A = 1/12 \pi r^2 = 1/12 \times 400 \times 24 \times 12 = 70043 \text{ in}^2$$

$$\text{Total Area} / A = \frac{113.09}{70043} = 0.001612 \text{ in}^2$$

The $\frac{1}{2}$ " ϕ 2024 spud is the strongest. Area = .625 sq.in.

Section P = 7.

Free Steel = 1/4 in. (proportional limit)

The $\frac{1}{2}$ " ϕ 2024 spud is the strongest

Steel to tie first to counterfort.

Section P = 8.

SI = 0" above the base.

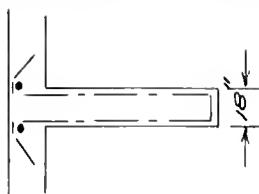
This pressure = 570 lb./sq.in. (calculated)

Proportion of areas of columns = 1 - 1/12 = 0.833

Steel regular size 1/2" x 12" counterfort spud area

$$\frac{3040}{20000} = .152 \text{ sq.in.}$$

Find force which must be applied to hold road.



$$\frac{F}{A} = 0.400 \text{ pounds per square inch}$$

the 7/8" plate has thickness
8" and width 10".

In order to develop the full strength of the road, the length must be 20.0 diameters. The length would be $20 \times 7\frac{1}{8} = 145\frac{5}{8}$ " long. This is for 1000 square inches. Since the rods are long 25 feet 10 1/2 inches. Length of counterfort at this point = $1 - 6"$. Road will be neck on road or back of counterfort. The two side rods will be 10 1/2 inches apart.

$$\text{Friction } F = 72$$

$$\text{The pressure} = 320 \text{ lb}$$

$$\text{Reaction of road on counterfort} = 320 \times 60" = 4160$$

$$\text{Total reaction from base + counterfort} = \frac{4480}{1.3333} \\ = .330 \text{ square in.}$$

Area road to be 33 square in.

$$\underline{330} = .330 \text{ square in.}$$

The 3/8" plate has thickness 8" = 10.52 "



SECTION C - C.

Unit pressure = 40000

Reaction at base = 11.4 kN = 11400 N

3300 = .214 sq.in steel required
approx.

Link road to trumpet.

$$\frac{P}{A} = 0.214 \text{ sq.in per kN}$$

Use 5/8" ϕ rods 1.5 m & 1.65" plates.

Total load on 1 m. run = 20100

Total weight of various items below :-

$$= (3 \times 10.5 \times 100 + 24.0 \times 1.65 \times 100) = \\ (3150 + 4032) = 7188.2 \text{ kg}$$

Total reaction = $\frac{7188.2}{1.25} = 5750.56 \text{ kN}$

Hold up plate.

The 13 x 7" ϕ rods = $13 \times 7 \times 1.65 = 11.02 \text{ m}$

Area of 7x13 in block of counterfort.

At the base:-

$$\therefore \frac{167 + 330 \times 20.5}{2} \times 2.7 \times 12 = 2163600$$

$$A = 2163600 \quad d = 3.75$$

$$A = \frac{2163600}{15000 \times 1.27 \times 3.75 \times 12} = \frac{2163600}{2160000} = 1.00 \text{ sq.m.}$$

Counterfort = 12" \times 7" \times 1.65 = 1.82 sq.m. for
one 12000.

Tr. 14. 1925-1926. 1 - " 1926 just as 1925.

$$= \frac{500 + 167}{2} = 333.5 \rightarrow 33.3 \times 10^{-3} \text{ m}^3 \text{ s}^{-1}$$

二二

$$\frac{d}{ds} \frac{1}{jd} = \frac{\cancel{1} \cdot \cancel{jd} - \cancel{j} \cdot \cancel{d}}{\cancel{jd} \cdot \cancel{jd}} = \frac{-jd}{jd \cdot jd} = \frac{-jd}{jd^2} = \frac{-j}{jd} = \frac{1}{jd} \cdot (-j)$$

7733 2" 3" role = 1" 2" 3" 4" 5" 6" 7" 8" 9" 10" 11" 12"

Two more could add to the total to 1000.

Cent 125 m = 1000' 1000' 1000' 1000'

1
3
3
3

— 1 —

• 1950-51 • 1951-52 • 1952-53 • 1953-54 • 1954-55 • 1955-56 • 1956-57 • 1957-58 • 1958-59 • 1959-60 • 1960-61 • 1961-62 • 1962-63 • 1963-64 • 1964-65 • 1965-66 • 1966-67 • 1967-68 • 1968-69 • 1969-70 • 1970-71 • 1971-72 • 1972-73 • 1973-74 • 1974-75 • 1975-76 • 1976-77 • 1977-78 • 1978-79 • 1979-80 • 1980-81 • 1981-82 • 1982-83 • 1983-84 • 1984-85 • 1985-86 • 1986-87 • 1987-88 • 1988-89 • 1989-90 • 1990-91 • 1991-92 • 1992-93 • 1993-94 • 1994-95 • 1995-96 • 1996-97 • 1997-98 • 1998-99 • 1999-2000 • 2000-2001 • 2001-2002 • 2002-2003 • 2003-2004 • 2004-2005 • 2005-2006 • 2006-2007 • 2007-2008 • 2008-2009 • 2009-2010 • 2010-2011 • 2011-2012 • 2012-2013 • 2013-2014 • 2014-2015 • 2015-2016 • 2016-2017 • 2017-2018 • 2018-2019 • 2019-2020 • 2020-2021 • 2021-2022 • 2022-2023 • 2023-2024

Consequently, the *in vitro* growth of *C. albicans* was inhibited by the presence of the *in vivo* metabolites of *S. faecalis*.

Flight CO₂ Counting Points

$$(\text{C}_6H_5)_2N + \text{CH}_3COCl \rightarrow (\text{C}_6H_5)_2N\text{COCl} + \text{HCl}$$

3. *Urtica dioica* L. (Urticaceae) - Common Nettle

“我就是想說，你這個人，真該死！你這個人，真該死！”

$$\begin{aligned} & \underline{0.5 \times 1.8 \times 150)0.000 + (7.0 \times 2.0 \times 150)0.000} \\ & + \underline{(1.5 \times 20.0 \times 150)0.000 + (4.0 \times 2.0 \times 150)0.000 = } \end{aligned}$$

$$= \frac{3570 + 2100 + 5400 + 4740}{11070} = \frac{15710}{11070} = 1.42$$

The point of application of the right ventricular kick

WILHELMUS VAN DER HORST, BORN IN 1850, IS THE FATHER OF



Total Vertical Force on Tonne on Appx. 1000.

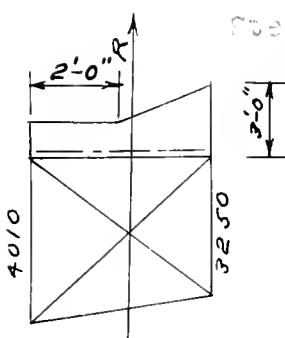
$$\frac{(1.3 \times 1000 - 100) \times .40 + (1.3 \times 1000 \times 100) \times .10}{\text{Total Vertical Weight}} + (11070 \times 10.58) + (3000 \times 7.00) + 1000 + 1000$$

$$= \frac{501790}{57945} = 8.90 \text{ feet}^2.$$

Horizontal force = 11070 lbs acts 3.0 feet above
the base. The resultant will make 30° with the mid-
dle third. Therefore, angle
centrifugally = 3.65 feet.

Pressure Head = $\frac{57945(1 + 1.3 \times 0.65)}{17} = 140.1 \text{ ft.}$

Pressure Head = $\frac{57945(1 + 1.3 \times 0.65)}{17} = 167.1 \text{ ft.}$



Soil at 50% of T.H.L.

$$\text{Soil} = \frac{167.1 - 140.1}{2} \times .50 \\ = 16.5 \text{ ft.}$$

Depth req'd for sand equals

$$\frac{16.5}{1.3 \times .65} = 70.1 \text{ inches}$$

Sand section S1 = 4" and slope 10:1 point 2' ft.
from the front.

$$\text{F.O.S.} = \frac{160.1 + 32.50 \times 1.3 \times 5.0 \times 10}{2} = 150.11 \text{ ft.}$$

$$150.11 - 1 = 149.11 \text{ ft.}$$



$$A = \frac{w}{\sigma_{sd}} = \frac{30000}{15000 k \cdot .37 \times 61} = 3.97 \text{ in}^2 \text{ per side}$$

Use 3/8" #6 round spaced 1" centers.

Steel in base.

Concrete beam from counter top to center of first 12" wide. Maximum load = $w = 12.5 \times 100 = 1250$

$$A = 1/12 \pi r^2 = 1/12 \times \pi \times 600 \times 31 \times 12 = 1015 \text{ in}^2$$

$$\text{Area steel required} = \frac{A}{\sigma_{sd}} = \frac{125000}{15000 k \cdot .37 \times 61} = 5.764 \text{ in}^2$$

Use 7/16" square rods spaced 3" centers for first 5 feet from E. side wall.

For next 3 1/2 ft to back of vertical wall:

$$w = 12.5 \times 100 = 1250 \text{ #}$$

$$A = 1/12 \pi r^2 = 1/12 \times \pi \times 1750 \times 14 \times 12 = 1062 \text{ in}^2$$

$$\text{Area steel required} = \frac{A}{\sigma_{sd}} = \frac{125000}{15000 k \cdot .37 \times 61} = 5.764 \text{ in}^2$$

Use 7/16" square rods spaced 3" centers to back of vertical wall.

Rods in Upper Part of Base.

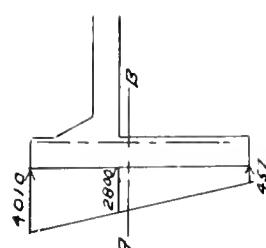
$$P = 10k \times 1 = 10.$$

Unit pressure due to eccentricity of central upward of base =

2660 # per square inch.

Unit pressure due to eccentricity at section (A - C) = $12.5 \times 100 = 1250$

at section (A - C) = $12.5 \times 100 = 1250$





$\alpha = 75^\circ - 45^\circ = 30^\circ$ from horizontal.

$$L = 1/12 \times 12 = 1, \text{ and } 1200 \text{ in. } L = 100 \text{ ft.}$$

$$\text{Area of trapezoid} = \frac{w_1 + w_2}{2} \times L = \frac{1200 + 100}{2} \times 100 = 65000 \text{ ft.}^2$$

$\therefore 65000 \text{ ft.}^2 \times 0.0001 = 6.5 \text{ acre ft.}$ (approx.)

Surface area of dam = 1000 ft. \times 100 ft. = 100000 ft. \times ft.

Section 6" = 6" \times 60 ft. \times 100 ft. = 36000 ft. \times ft.

Total horizontal projection = 100 ft. \times

$$w_{\text{max}} = 100000 / 100 = 1000 \text{ ft.}$$

$$l = \frac{w_1 + w_2}{2} = \frac{1200 + 100}{2} = 650 \text{ ft. min.}$$

The 1" square foot capacity = 100 ft. \times ft.

Area = 7.50 ft. \times ft.

Surface area of dam = 100000 ft. \times ft.

$$\text{Total per sec. output} = \frac{100000 \times 100}{7.50} = 133333 \text{ ft.}^3$$

Therefore total horizontal projection = 100000 ft. \times ft.

$$\frac{133333}{100000} \times 100 = 13.33 \text{ ft.}$$

$$w_{\text{min}} = 13.33 \times 100 = 1333 \text{ ft.}$$

$$l = \frac{w_1 + w_2}{2} = \frac{1333 + 100}{2} = 666.5 \text{ ft. min.}$$

3.04 = 3.00 \pm 0.02 ft. min. (allowable variation)

The 12" square foot capacity = 100 ft. \times 100 ft. \times ft.

(6) Effect of water head on dam structure.



2.73 $\times 10^{-2} \text{ m}^2 = 10^{-2} \text{ m}^2 = 10^{-2} \text{ m}^2 = 10^{-2} \text{ m}^2$

$$= 1/2 \times 10^{-2} = 1/2 \times 10^{-2} = 1/2 \times 10^{-2}$$

$$\lambda = \frac{1}{2} = \frac{1}{2} = \frac{1}{2} = 0.5 \text{ m}$$

$$v = 2/2 = 1 \text{ m/s}$$



Test Soil 6" dia = 100% penetration
Total weight to be on top of soil = 1000 lbs
traction force which is 1000 lbs / 100%
= 1000 lbs per 100%

$$(10000 + 1000 \text{ lbs}) \times 0.005 = 50000 \text{ lbs} \text{ per } 100\% \text{ area}$$
$$(10000 + 1000 \text{ lbs}) \times 0.005 = 100000 \text{ lbs per } 100\% \text{ area}$$

$$100000 \times 0.005 \text{ lbs per } 100\% \text{ area} = 50000 \text{ lbs per } 100\% \text{ area}$$
$$= 50000 "$$

$$\frac{1}{2} \times 0.1 \times 0.005 \text{ lbs} = 0.00025 \text{ lbs}$$

$$\frac{1}{2} \times 0.1 \times 0.005 \text{ lbs} = 0.00025 \text{ lbs}$$

$$50000 + 100000 + 50000 + 10000 + 0.00025 = 200000.00025$$

the maximum reaction can be on 100% area.

The shear plane will pass through 100% of the sample.

$$= 100000 \text{ lbs per } 100\% \text{ area}$$

$$= \text{Shear of mass} = 100000 "$$

$$= 100000 \text{ lbs per } 100\% \text{ area}$$

Total weight to be on top of soil = 1000 lbs / 100% =

$$= 100000 + 100000 = 200000$$

$$= 100000 \text{ lbs per } 100\% \text{ area}$$

$$= \text{shear of mass} = 100000 "$$

$$= \text{Shear of mass} = 100000 - 0 "$$

$$= \text{penetration of last load} = 2"$$



$\tau = \tau_0 \ln(\omega/\omega_0) + \tau_1$ or $\tau = \tau_0 \ln(\omega/\omega_0) - \tau_2$

$$\omega = \omega_0 e^{\pm i(\tau - \tau_0 \ln(\omega_0/\omega))} = \frac{\omega_0}{\omega} e^{\pm i(\tau - \tau_0)}$$

$\omega = \omega_0 e^{\pm i(\tau - \tau_0)}$ is the dispersion relation for the wave.

Estimate of Total Concrete Volume

Volume of concrete in each slab =

$$= \pi (a_1 + a_2)$$

$$= \pi (12 + 20.7) + (2 + 2) = 125 \text{ cu ft.}$$

$$= (125 \times 70.85) + (2 \times 20.85) = 8800 \text{ cu ft.}$$

$$V = \frac{\pi}{2} (a_1^2 + a_2^2 + a_1 a_2) = 125 \times 103.27$$

= 6100 cubic feet or 20 cubic yards.

Concrete in walls =

$$= (1 \times 12.125 \times 75.85) + (2 \times 1 \times 7 \times 1.65)$$

$$= 1080 + 107.7 = 1187 \text{ cu ft.} = 39.9 \text{ cu yds.}$$

Volume of concrete in columns =

$$100 \times 50.5 = 5050.$$

Volume of concrete in floor joists =

$$1 \times 600 \times 2 = 1200 \text{ cu ft.}$$

The total volume of concrete will be available soon on Fort Det. The completed columns will be similar to the first and all members are signed. The cylindrical columns and rectangular cells were made similar to those of the first two projects.



Справочник по геометрии для 8-го класса

$$= \frac{1}{2} \left(\frac{1}{\lambda_1} + \frac{1}{\lambda_2} \right) = \frac{1}{2} \left(\frac{1}{\lambda_1} + \frac{1}{\lambda_2} \right)$$

Digitized by srujanika@gmail.com

10. The following table gives the number of hours worked by 1000 workers in a certain industry.

$$\text{Volumen Vakuum} = (1 - 15,7 \cdot 10^{-3}) = 18,2 \cdot 10^{-3}$$

$$\text{Cubic} = \pi \times \frac{D^2 + d^2}{4} \times 7.67 = 27.1 \text{ cubic ft.}$$
$$= 0.5 \text{ cubic yds.}$$

$$\text{Cylinders} = (\pi d^2 \times 1) + \frac{D^2 + d^2}{4} \times 7.67 = 11.2 \text{ cubic ft.} = 0.2 \text{ cubic yds.}$$

Countercurrent:

$$3 \times \frac{8+6}{2} \times 10.5 \times 7.2 = 111 \text{ cubic ft.} = 0.3 \text{ cubic yds.}$$

$$3 \times \frac{10.5+12}{2} \times 10.5 \times 7.2 = 157 \text{ cubic ft.} = 0.4 \text{ cubic yds.}$$

$$3 \times 6/2 \times 12.5 \times 7.2 = 135.0 \text{ cubic ft.} = 0.3 \text{ cubic yds.}$$

$$3 \times 4/2 \times 12.5 \times 7.2 = 72 \text{ cubic ft.} = 0.1 \text{ cubic yds.}$$

$$3 \times 2.5 \times 6.5 \times 7.2 = 45.0 \text{ cubic ft.} = 0.1 \text{ cubic yds.}$$

$$\text{Total} = 215.0 + 1.2 + 157.0 + 7.2 + 135.0 + 0.3$$

$$+ 5.0 + 0.7 + 0.2 = 507.1 \text{ cubic ft.} = 14.0 \text{ cu. yds.} \text{ per cut}$$

ment.

$$3 \times 80^2 \times 2 = 414.0 \text{ cubic yds.} \text{ per cut.} \text{ This is the volume of the excavation.}$$

Weight of earth excavated.

In figuring the weight of earth excavated we must consider the weight of the rock and the percentage of the blue plant and multiplied by the weight factor of 100 as given in the table of the Carnegie Steel Company.



The following are the dimensions of the
beam.

$$12 \times 16 = 192 \text{ in} \times 16 \text{ in} \quad 1.00 \text{ in} \times 100 \text{ in}$$

$$8 \times 16 = 128 \text{ in} \times 16 \text{ in} \quad 0.75 \text{ in} \times 100 \text{ in}$$

$$6 \times 16 = 96 \text{ in} \times 16 \text{ in} \quad 0.50 \text{ in} \times 100 \text{ in}$$

$$4 \times 16 = 64 \text{ in} \times 16 \text{ in} \quad 0.25 \text{ in} \times 100 \text{ in}$$

$$2 \times 16 = 32 \text{ in} \times 16 \text{ in} \quad 0.125 \text{ in} \times 100 \text{ in}$$

"Counterforted

$$\text{Rect. section} \quad 6 \times 16 = 96 \text{ in} \times 16 \text{ in}$$

$$\text{At Park} \quad 16 \times 20 \text{ in} \times 16 \text{ in} = 100 \text{ cu in}$$

$$6 \times 16 = 96 \text{ in} \times 16 \text{ in} = 0.15 \text{ cu in}$$

$$\text{Hor. steel} \quad 5 \times 160 = 800 = 160 \text{ in}$$

Wall:

$$\text{Rect. section} \quad 60 \times 17 = 660 = 66 \text{ cu in}$$

$$\text{Or. steel} \quad 27 \times 20 = 540 = 270 \text{ cu in}$$

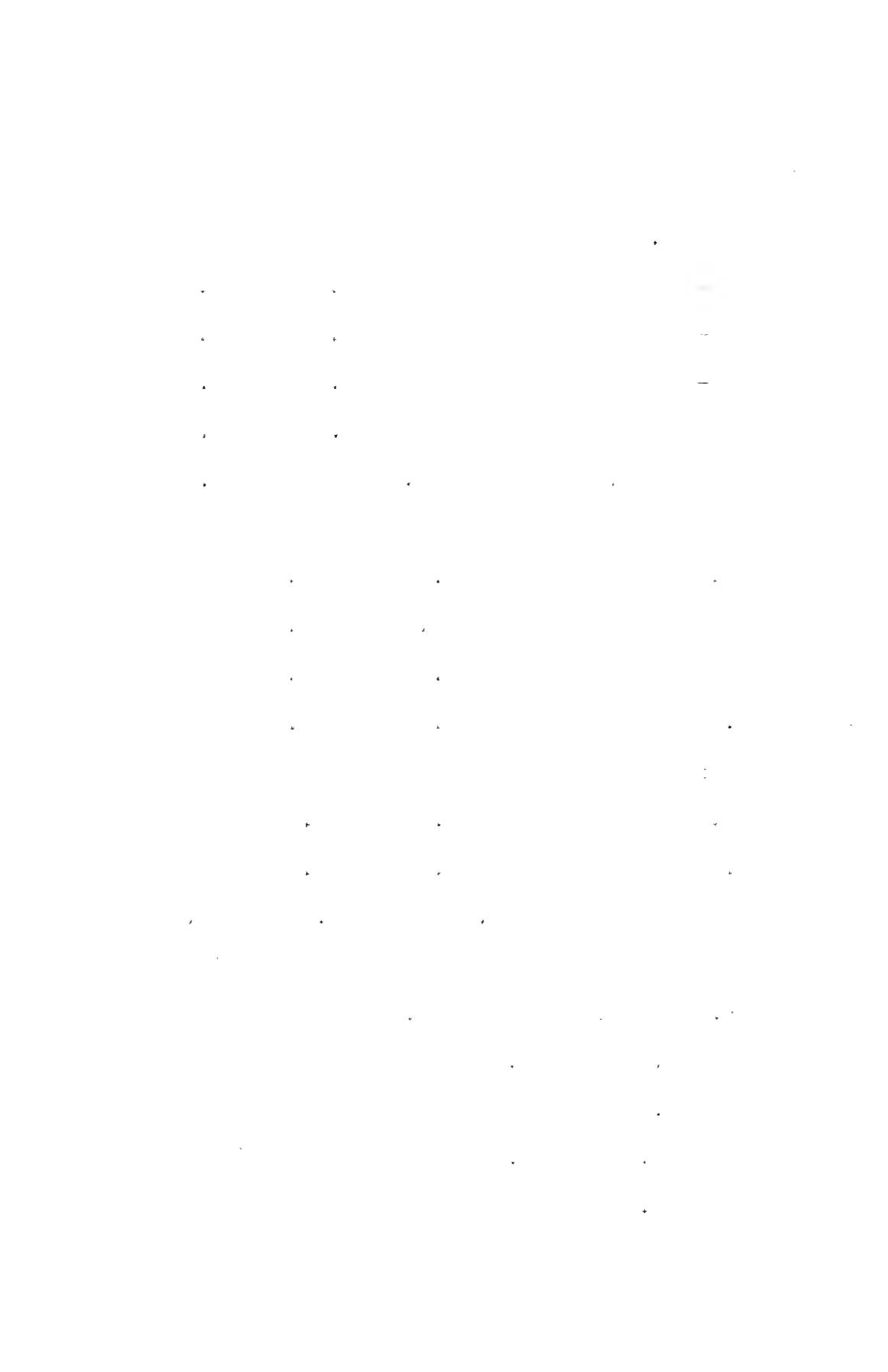
$$\text{Curb and parapet} = 22.3 \times 16 = 357 = 35.7 \text{ cu in}$$

total weight of one cubic foot of aluminum type 2
is 260.40 lb weight of one cu in.

$$1 \times 8068.4 = 8068.4 \text{ lb/in. weight of one cu in. aluminum}$$

counterfort.

$$5 \times 11370.8 = 56854.0 \text{ weight of one cu in. aluminum.}$$



Cost of Abutments.

<u>Material</u>	<u>Cost per cu.yd.</u>
Cement @ " 3.73	4.40
Sand @ ".50	.53
Gravel @ ".50	.48
Lumber @ " 35.53	.86
Piles @ ".20	.06
Machinery	.10
Wire and nails	.10
Lubricating oil	.01
Fuel	<u>.10</u> <u>6.54</u>

<u>Labor</u>	<u>Cost per cu.yd.</u>
Excavation for foundation	.34
Building and removing forms	.57
Driving piles in foundation	.11
Placing steel reinforcement	.16
Mixing concrete	.38
Placing concrete	.27
Pumping water	.03
Cleaning and storing machines	<u>.10</u> <u>1.56</u> <u>5.54</u> <u>8.40</u>
Total material & labor	



Total cost of 2000 ft² of brick = \$ 100000
= 200000.00 ft² = $\frac{1}{10}$ acre

Total cost of 2000 ft² of brick = 100000 + 100000.00 =
= 200000.00

Total cost of 1250 ft² of brick =

Using 100000 ft² of brick = 100000/10 = 10000 ft²

10000 ft² = 1 acre

Total cost of 1250 ft² of brick = 100000/10
= 100000/10 = 10000 ft² = 1 acre

Total cost of 1250 ft² of brick = 100000/10
= 10000 ft² = 1 acre

Total cost of 1250 ft² of brick = 100000/10

Total cost of 1250 ft² of brick = 100000/10

100000.00 + 100000.00 = 200000.00



Estimate of Cost of Superstructure.

Weight of Steel in 75'-0" Span.

The following weights are for one half
of one truss.

Member.

Upper Chord.	Length	Weight.
2 angles 6" x 4" x 5/8"	30'-0"	1200.00 #
1 Cov. Pl. 12" x 5/8"	30'-0"	612.00
1 Splice Pl. 12" x 5/8"	3'-0"	76.50
1 Splice Pl. 12" x 1/2"	18'-0"	20.40

End Post.

2 angles 6" x 4" x 5/8"	10'-0"	400.00
1 Cov. Pl. 12" x 1/2"	10'-0"	204.00

Lower Chord.

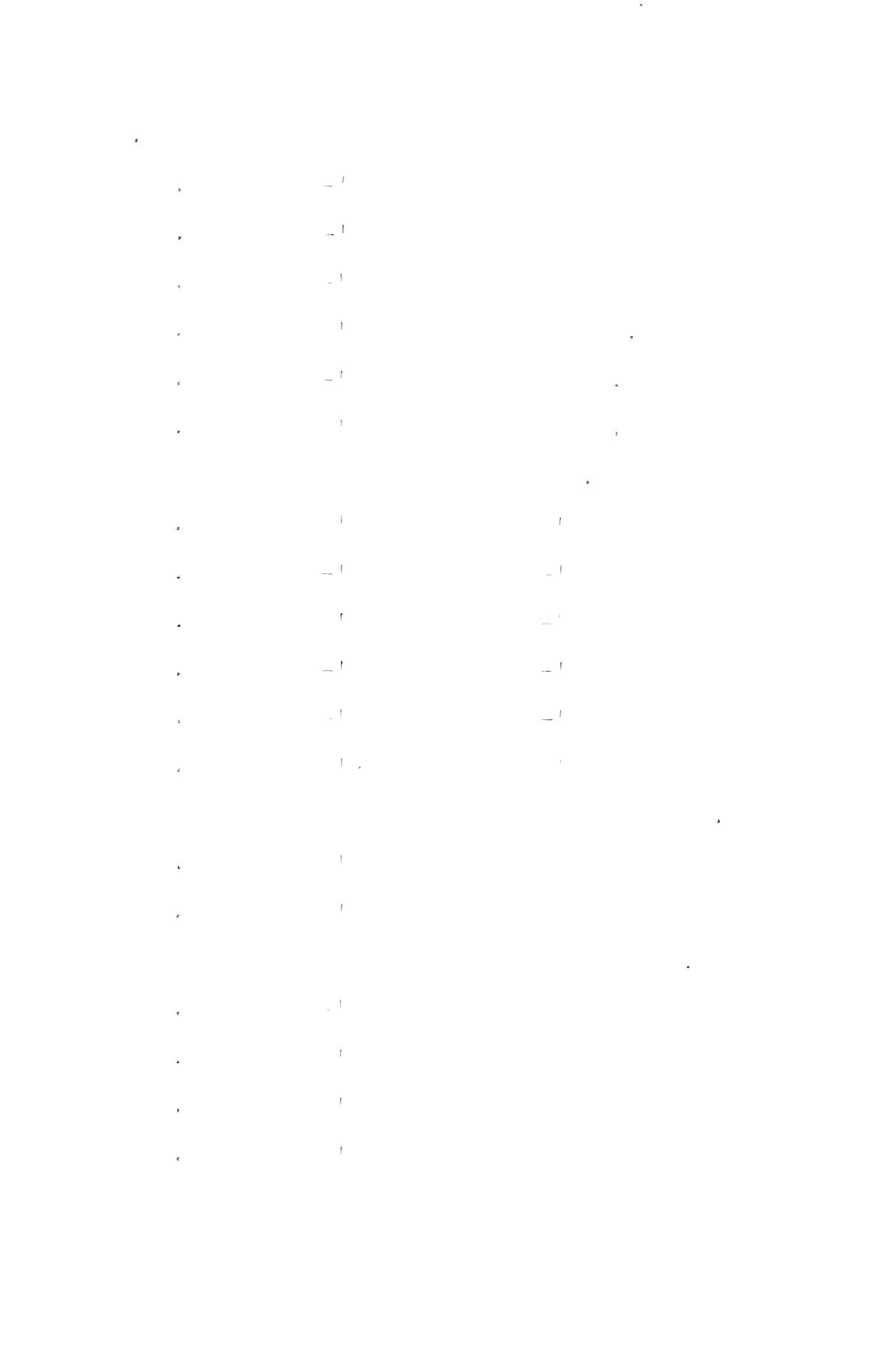
3 angles 6" x 4" x 7/8"	37'-0"	2040.00
1 Splice Pl. 12" x 5/8"	0'-10"	10.75
1 Splice Pl. 12" x 3/8"	3'-1"	42.17
1 Splice Pl. 12" x 3/8"	0'-6"	37.80

Verticals.

4 angles 4" x 3" x 5/16"	6'-6"	197.60
4 Batten Pl. 12" x 1/2"	0'-5"	34.00
4 Ext. Ang. 4" x 3" x 5/16"	3'-7"	63.60



	Length	Width
Diagonals		
2 angles 4" x 3" x 11/16"	14-1"	222.40
2 angles 4" x 3 $\frac{1}{2}$ " x 1 $\frac{1}{2}$ "	14-0"	226.40
2 angles 3" x 3" x 1 $\frac{1}{2}$ "	14-0"	101.00
2 clip angles 1" x 1 $\frac{1}{2}$ " x 1 $\frac{1}{2}$ "	11-0"	62.60
4 batten 1 $\frac{1}{2}$ " x 1 $\frac{1}{2}$ " x 1 $\frac{1}{4}$ "	11-0"	12.00
4 batten 1 $\frac{1}{2}$ " x 1 $\frac{1}{2}$ " x 4"	11-0"	51.00
Brackets Plastic		
2 plates 1 $\frac{1}{2}$ " x 11-10"	11-0"	100.00
2 plates 1 $\frac{1}{2}$ " x 11-10"	11-0"	737.00
2 plates 1 $\frac{1}{2}$ " x 11-9"	11-0"	312.70
2 plates 1 $\frac{1}{2}$ " x 11-1"	11-0"	16.00
2 plates 1 $\frac{1}{2}$ " x 11-10"	11-0"	760.00
2 plates 1 $\frac{1}{2}$ " x 11-9"	11-0"	109.70
Caps		
2 angles 5 $\frac{1}{2}$ " x 3 $\frac{1}{2}$ " x 1 $\frac{1}{2}$ "	11-0"	74.00
2 plates 1 $\frac{1}{2}$ " x 10"	11-0"	21.40
Washing		
65 @ 1 $\frac{1}{2}$ " x 3 $\frac{1}{2}$ "	61-0"	310.00
65 @ 2" x 1 $\frac{1}{2}$ "	61-0"	65.62
20 @ 1 $\frac{1}{4}$ " x 1 $\frac{1}{4}$ "	21-0"	27.10
70 @ 1 $\frac{1}{4}$ " x 1 $\frac{1}{4}$ "	21-0"	168.70



7.25 - 0.25 = 7.00
 7.00 + 1.00 = 8.00
 8.00 - 0.25 = 7.75
 7.75 + 1.00 = 8.75
 8.75 - 0.25 = 8.50
 8.50 + 1.00 = 9.50
 9.50 - 0.25 = 9.25
 9.25 + 1.00 = 10.25
 10.25 - 0.25 = 10.00
 10.00 + 1.00 = 11.00
 11.00 - 0.25 = 10.75
 10.75 + 1.00 = 11.75
 11.75 - 0.25 = 11.50
 11.50 + 1.00 = 12.50
 12.50 - 0.25 = 12.25
 12.25 + 1.00 = 13.25
 13.25 - 0.25 = 13.00
 13.00 + 1.00 = 14.00
 14.00 - 0.25 = 13.75
 13.75 + 1.00 = 14.75
 14.75 - 0.25 = 14.50
 14.50 + 1.00 = 15.50
 15.50 - 0.25 = 15.25
 15.25 + 1.00 = 16.25
 16.25 - 0.25 = 16.00
 16.00 + 1.00 = 17.00
 17.00 - 0.25 = 16.75
 16.75 + 1.00 = 17.75
 17.75 - 0.25 = 17.50
 17.50 + 1.00 = 18.50
 18.50 - 0.25 = 18.25
 18.25 + 1.00 = 19.25
 19.25 - 0.25 = 19.00
 19.00 + 1.00 = 20.00
 20.00 - 0.25 = 19.75
 19.75 + 1.00 = 20.75
 20.75 - 0.25 = 20.50
 20.50 + 1.00 = 21.50
 21.50 - 0.25 = 21.25
 21.25 + 1.00 = 22.25
 22.25 - 0.25 = 22.00
 22.00 + 1.00 = 23.00
 23.00 - 0.25 = 22.75
 22.75 + 1.00 = 23.75
 23.75 - 0.25 = 23.50
 23.50 + 1.00 = 24.50
 24.50 - 0.25 = 24.25
 24.25 + 1.00 = 25.25
 25.25 - 0.25 = 25.00
 25.00 + 1.00 = 26.00
 26.00 - 0.25 = 25.75
 25.75 + 1.00 = 26.75
 26.75 - 0.25 = 26.50
 26.50 + 1.00 = 27.50
 27.50 - 0.25 = 27.25
 27.25 + 1.00 = 28.25
 28.25 - 0.25 = 28.00
 28.00 + 1.00 = 29.00
 29.00 - 0.25 = 28.75
 28.75 + 1.00 = 29.75
 29.75 - 0.25 = 29.50
 29.50 + 1.00 = 30.50
 30.50 - 0.25 = 30.25
 30.25 + 1.00 = 31.25
 31.25 - 0.25 = 31.00
 31.00 + 1.00 = 32.00
 32.00 - 0.25 = 31.75
 31.75 + 1.00 = 32.75
 32.75 - 0.25 = 32.50
 32.50 + 1.00 = 33.50
 33.50 - 0.25 = 33.25
 33.25 + 1.00 = 34.25
 34.25 - 0.25 = 34.00
 34.00 + 1.00 = 35.00
 35.00 - 0.25 = 34.75
 34.75 + 1.00 = 35.75
 35.75 - 0.25 = 35.50
 35.50 + 1.00 = 36.50
 36.50 - 0.25 = 36.25
 36.25 + 1.00 = 37.25
 37.25 - 0.25 = 37.00
 37.00 + 1.00 = 38.00
 38.00 - 0.25 = 37.75
 37.75 + 1.00 = 38.75
 38.75 - 0.25 = 38.50
 38.50 + 1.00 = 39.50
 39.50 - 0.25 = 39.25
 39.25 + 1.00 = 40.25
 40.25 - 0.25 = 40.00
 40.00 + 1.00 = 41.00
 41.00 - 0.25 = 40.75
 40.75 + 1.00 = 41.75
 41.75 - 0.25 = 41.50
 41.50 + 1.00 = 42.50
 42.50 - 0.25 = 42.25
 42.25 + 1.00 = 43.25
 43.25 - 0.25 = 43.00
 43.00 + 1.00 = 44.00
 44.00 - 0.25 = 43.75
 43.75 + 1.00 = 44.75
 44.75 - 0.25 = 44.50
 44.50 + 1.00 = 45.50
 45.50 - 0.25 = 45.25
 45.25 + 1.00 = 46.25
 46.25 - 0.25 = 46.00
 46.00 + 1.00 = 47.00
 47.00 - 0.25 = 46.75
 46.75 + 1.00 = 47.75
 47.75 - 0.25 = 47.50
 47.50 + 1.00 = 48.50
 48.50 - 0.25 = 48.25
 48.25 + 1.00 = 49.25
 49.25 - 0.25 = 49.00
 49.00 + 1.00 = 50.00
 50.00 - 0.25 = 49.75
 49.75 + 1.00 = 50.75
 50.75 - 0.25 = 50.50
 50.50 + 1.00 = 51.50
 51.50 - 0.25 = 51.25
 51.25 + 1.00 = 52.25
 52.25 - 0.25 = 52.00
 52.00 + 1.00 = 53.00
 53.00 - 0.25 = 52.75
 52.75 + 1.00 = 53.75
 53.75 - 0.25 = 53.50
 53.50 + 1.00 = 54.50
 54.50 - 0.25 = 54.25
 54.25 + 1.00 = 55.25
 55.25 - 0.25 = 55.00
 55.00 + 1.00 = 56.00
 56.00 - 0.25 = 55.75
 55.75 + 1.00 = 56.75
 56.75 - 0.25 = 56.50
 56.50 + 1.00 = 57.50
 57.50 - 0.25 = 57.25
 57.25 + 1.00 = 58.25
 58.25 - 0.25 = 58.00
 58.00 + 1.00 = 59.00
 59.00 - 0.25 = 58.75
 58.75 + 1.00 = 59.75
 59.75 - 0.25 = 59.50
 59.50 + 1.00 = 60.50
 60.50 - 0.25 = 60.25
 60.25 + 1.00 = 61.25
 61.25 - 0.25 = 61.00
 61.00 + 1.00 = 62.00
 62.00 - 0.25 = 61.75
 61.75 + 1.00 = 62.75
 62.75 - 0.25 = 62.50
 62.50 + 1.00 = 63.50
 63.50 - 0.25 = 63.25
 63.25 + 1.00 = 64.25
 64.25 - 0.25 = 64.00
 64.00 + 1.00 = 65.00
 65.00 - 0.25 = 64.75
 64.75 + 1.00 = 65.75
 65.75 - 0.25 = 65.50
 65.50 + 1.00 = 66.50
 66.50 - 0.25 = 66.25
 66.25 + 1.00 = 67.25
 67.25 - 0.25 = 67.00
 67.00 + 1.00 = 68.00
 68.00 - 0.25 = 67.75
 67.75 + 1.00 = 68.75
 68.75 - 0.25 = 68.50
 68.50 + 1.00 = 69.50
 69.50 - 0.25 = 69.25
 69.25 + 1.00 = 70.25
 70.25 - 0.25 = 70.00
 70.00 + 1.00 = 71.00
 71.00 - 0.25 = 70.75
 70.75 + 1.00 = 71.75
 71.75 - 0.25 = 71.50
 71.50 + 1.00 = 72.50
 72.50 - 0.25 = 72.25
 72.25 + 1.00 = 73.25
 73.25 - 0.25 = 73.00
 73.00 + 1.00 = 74.00
 74.00 - 0.25 = 73.75
 73.75 + 1.00 = 74.75
 74.75 - 0.25 = 74.50
 74.50 + 1.00 = 75.50
 75.50 - 0.25 = 75.25
 75.25 + 1.00 = 76.25
 76.25 - 0.25 = 76.00
 76.00 + 1.00 = 77.00
 77.00 - 0.25 = 76.75
 76.75 + 1.00 = 77.75
 77.75 - 0.25 = 77.50
 77.50 + 1.00 = 78.50
 78.50 - 0.25 = 78.25
 78.25 + 1.00 = 79.25
 79.25 - 0.25 = 79.00
 79.00 + 1.00 = 80.00
 80.00 - 0.25 = 79.75
 79.75 + 1.00 = 80.75
 80.75 - 0.25 = 80.50
 80.50 + 1.00 = 81.50
 81.50 - 0.25 = 81.25
 81.25 + 1.00 = 82.25
 82.25 - 0.25 = 82.00
 82.00 + 1.00 = 83.00
 83.00 - 0.25 = 82.75
 82.75 + 1.00 = 83.75
 83.75 - 0.25 = 83.50
 83.50 + 1.00 = 84.50
 84.50 - 0.25 = 84.25
 84.25 + 1.00 = 85.25
 85.25 - 0.25 = 85.00
 85.00 + 1.00 = 86.00
 86.00 - 0.25 = 85.75
 85.75 + 1.00 = 86.75
 86.75 - 0.25 = 86.50
 86.50 + 1.00 = 87.50
 87.50 - 0.25 = 87.25
 87.25 + 1.00 = 88.25
 88.25 - 0.25 = 88.00
 88.00 + 1.00 = 89.00
 89.00 - 0.25 = 88.75
 88.75 + 1.00 = 89.75
 89.75 - 0.25 = 89.50
 89.50 + 1.00 = 90.50
 90.50 - 0.25 = 90.25
 90.25 + 1.00 = 91.25
 91.25 - 0.25 = 91.00
 91.00 + 1.00 = 92.00
 92.00 - 0.25 = 91.75
 91.75 + 1.00 = 92.75
 92.75 - 0.25 = 92.50
 92.50 + 1.00 = 93.50
 93.50 - 0.25 = 93.25
 93.25 + 1.00 = 94.25
 94.25 - 0.25 = 94.00
 94.00 + 1.00 = 95.00
 95.00 - 0.25 = 94.75
 94.75 + 1.00 = 95.75
 95.75 - 0.25 = 95.50
 95.50 + 1.00 = 96.50
 96.50 - 0.25 = 96.25
 96.25 + 1.00 = 97.25
 97.25 - 0.25 = 97.00
 97.00 + 1.00 = 98.00
 98.00 - 0.25 = 97.75
 97.75 + 1.00 = 98.75
 98.75 - 0.25 = 98.50
 98.50 + 1.00 = 99.50
 99.50 - 0.25 = 99.25
 99.25 + 1.00 = 100.25
 100.25 - 0.25 = 100.00
 100.00 + 1.00 = 101.00
 101.00 - 0.25 = 100.75
 100.75 + 1.00 = 101.75
 101.75 - 0.25 = 101.50
 101.50 + 1.00 = 102.50
 102.50 - 0.25 = 102.25
 102.25 + 1.00 = 103.25
 103.25 - 0.25 = 103.00
 103.00 + 1.00 = 104.00
 104.00 - 0.25 = 103.75
 103.75 + 1.00 = 104.75
 104.75 - 0.25 = 104.50
 104.50 + 1.00 = 105.50
 105.50 - 0.25 = 105.25
 105.25 + 1.00 = 106.25
 106.25 - 0.25 = 106.00
 106.00 + 1.00 = 107.00
 107.00 - 0.25 = 106.75
 106.75 + 1.00 = 107.75
 107.75 - 0.25 = 107.50
 107.50 + 1.00 = 108.50
 108.50 - 0.25 = 108.25
 108.25 + 1.00 = 109.25
 109.25 - 0.25 = 109.00
 109.00 + 1.00 = 110.00
 110.00 - 0.25 = 109.75
 109.75 + 1.00 = 110.75
 110.75 - 0.25 = 110.50
 110.50 + 1.00 = 111.50
 111.50 - 0.25 = 111.25
 111.25 + 1.00 = 112.25
 112.25 - 0.25 = 112.00
 112.00 + 1.00 = 113.00
 113.00 - 0.25 = 112.75
 112.75 + 1.00 = 113.75
 113.75 - 0.25 = 113.50
 113.50 + 1.00 = 114.50
 114.50 - 0.25 = 114.25
 114.25 + 1.00 = 115.25
 115.25 - 0.25 = 115.00
 115.00 + 1.00 = 116.00
 116.00 - 0.25 = 115.75
 115.75 + 1.00 = 116.75
 116.75 - 0.25 = 116.50
 116.50 + 1.00 = 117.50
 117.50 - 0.25 = 117.25
 117.25 + 1.00 = 118.25
 118.25 - 0.25 = 118.00
 118.00 + 1.00 = 119.00
 119.00 - 0.25 = 118.75
 118.75 + 1.00 = 119.75
 119.75 - 0.25 = 119.50
 119.50 + 1.00 = 120.50
 120.50 - 0.25 = 120.25
 120.25 + 1.00 = 121.25
 121.25 - 0.25 = 121.00
 121.00 + 1.00 = 122.00
 122.00 - 0.25 = 121.75
 121.75 + 1.00 = 122.75
 122.75 - 0.25 = 122.50
 122.50 + 1.00 = 123.50
 123.50 - 0.25 = 123.25
 123.25 + 1.00 = 124.25
 124.25 - 0.25 = 124.00
 124.00 + 1.00 = 125.00
 125.00 - 0.25 = 124.75
 124.75 + 1.00 = 125.75
 125.75 - 0.25 = 125.50
 125.50 + 1.00 = 126.50
 126.50 - 0.25 = 126.25
 126.25 + 1.00 = 127.25
 127.25 - 0.25 = 127.00
 127.00 + 1.00 = 128.00
 128.00 - 0.25 = 127.75
 127.75 + 1.00 = 128.75
 128.75 - 0.25 = 128.50
 128.50 + 1.00 = 129.50
 129.50 - 0.25 = 129.25
 129.25 + 1.00 = 130.25
 130.25 - 0.25 = 130.00
 130.00 + 1.00 = 131.00
 131.00 - 0.25 = 130.75
 130.75 + 1.00 = 131.75
 131.75 - 0.25 = 131.50
 131.50 + 1.00 = 132.50
 132.50 - 0.25 = 132.25
 132.25 + 1.00 = 133.25
 133.25 - 0.25 = 133.00
 133.00 + 1.00 = 134.00
 134.00 - 0.25 = 133.75
 133.75 + 1.00 = 134.75
 134.75 - 0.25 = 134.50
 134.50 + 1.00 = 135.50
 135.50 - 0.25 = 135.25
 135.25 + 1.00 = 136.25
 136.25 - 0.25 = 136.00
 136.00 + 1.00 = 137.00
 137.00 - 0.25 = 136.75
 136.75 + 1.00 = 137.75
 137.75 - 0.25 = 137.50
 137.50 + 1.00 = 138.50
 138.50 - 0.25 = 138.25
 138.25 + 1.00 = 139.25
 139.25 - 0.25 = 139.00
 139.00 + 1.00 = 140.00
 140.00 - 0.25 = 139.75
 139.75 + 1.00 = 140.75
 140.75 - 0.25 = 140.50
 140.50 + 1.00 = 141.50
 141.50 - 0.25 = 141.25
 141.25 + 1.00 = 142.25
 142.25 - 0.25 = 142.00
 142.00 + 1.00 = 143.00
 143.00 - 0.25 = 142.75
 142.75 + 1.00 = 143.75
 143.75 - 0.25 = 143.50
 143.50 + 1.00 = 144.50
 144.50 - 0.25 = 144.25
 144.25 + 1.00 = 145.25
 145.25 - 0.25 = 145.00
 145.00 + 1.00 = 146.00
 146.00 - 0.25 = 145.75
 145.75 + 1.00 = 146.75
 146.75 - 0.25 = 146.50
 146.50 + 1.00 = 147.50
 147.50 - 0.25 = 147.25
 147.25 + 1.00 = 148.25
 148.25 - 0.25 = 148.00
 148.00 + 1.00 = 149.00
 149.00 - 0.25 = 148.75
 148.75 + 1.00 = 149.75
 149.75 - 0.25 = 149.50
 149.50 + 1.00 = 150.50
 150.50 - 0.25 = 150.25
 150.25 + 1.00 = 151.25
 151.25 - 0.25 = 151.00
 151.00 + 1.00 = 152.00
 152.00 - 0.25 = 151.75
 151.75 + 1.00 = 152.75
 152.75 - 0.25 = 152.50
 152.50 + 1.00 = 153.50
 153.50 - 0.25 = 153.25
 153.25 + 1.00 = 154.25
 154.25 - 0.25 = 154.00
 154.00 + 1.00 = 155.00
 155.00 - 0.25 = 154.75
 154.75 + 1.00 = 155.75
 155.75 - 0.25 = 155.50
 155.50 + 1.00 = 156.50
 156.50 - 0.25 = 156.25
 156.25 + 1.00 = 157.25
 157.25 - 0.25 = 157.00
 157.00 + 1.00 = 158.00
 158.00 - 0.25 = 157.75
 157.75 + 1.00 = 158.75
 158.75 - 0.25 = 158.50
 158.50 + 1.00 = 159.50
 159.50 - 0.25 = 159.25
 159.25 + 1.00 = 160.25
 160.25 - 0.25 = 160.00
 160.00 + 1.00 = 161.00
 161.00 - 0.25 = 160.75
 160.75 + 1.00 = 161.75
 161.75 - 0.25 = 161.50
 161.50 + 1.00 = 162.50
 162.50 - 0.25 = 162.25
 162.25 + 1.00 = 163.25
 163.25 - 0.25 = 163.00
 163.00 + 1.00 = 164.00
 164.00 - 0.25 = 163.75
 163.75 + 1.00 = 164.75
 164.75 - 0.25 = 164.50
 164.50 + 1.00 = 165.50
 165.50 - 0.25 = 165.25
 165.25 + 1.00 = 166.25
 166.25 - 0.25 = 166.00
 166.00 + 1.00 = 167.00
 167.00 - 0.25 = 166.75
 166.75 + 1.00 = 167.75
 167.75 - 0.25 = 167.50
 167.50 + 1.00 = 168.50
 168.50 - 0.25 = 168.25
 168.25 + 1.00 = 169.25
 169.25 - 0.25 = 169.00
 169.00 + 1.00 = 170.00
 170.00 - 0.25 = 169.75
 169.75 + 1.00 = 170.75
 170.75 - 0.25 = 170.50
 170.50 + 1.00 = 171.50
 171.50 - 0.25 = 171.25
 171.25 + 1.00 = 172.25
 172.25 - 0.25 = 172.00
 172.00 + 1.00 = 173.00
 173.00 - 0.25 = 172.75
 172.75 + 1.00 = 173.75
 173.75 - 0.25 = 173.50
 173.50 + 1.00 = 174.50
 174.50 - 0.25 = 174.25
 174.25 + 1.00 = 175.25
 175.25 - 0.25 = 175.00
 175.00 + 1.00 = 176.00
 176.00 - 0.25 = 175.75
 175.75 + 1.00 = 176.75
 176.75 - 0.25 = 176.50
 176.50 + 1.00 = 177.50
 177.50 - 0.25 = 177.25
 177.25 + 1.00 = 178.25
 178.25 - 0.25 = 178.00
 178.00 + 1.00 = 179.00
 179.00 - 0.25 = 178.75
 178.75 + 1.00 = 179.75
 179.75 - 0.25 = 179.50
 179.50 + 1.00 = 180.50
 180.50 - 0.25 = 180.25
 180.25 + 1.00 = 181.25
 181.25 - 0.25 = 181.00
 181.00 + 1.00 = 182.00
 182.00 - 0.25 = 181.75
 181.75 + 1.00 = 182.75
 182.75 - 0.25 = 182.50
 182.50 + 1.00 = 183.50
 183.50 - 0.25 = 183.25
 183.25 + 1.00 = 184.25
 184.25 - 0.25 = 184.00
 184.00 + 1.00 = 185.00
 185.00 - 0.25 = 184.75
 184.75 + 1.00 = 185.75
 185.75 - 0.25 = 185.50
 185.50 + 1.00 = 186.50
 186.50 - 0.25 = 186.25
 186.25 + 1.00 = 187.25
 187.25 - 0.25 = 187.00
 187.00 + 1.00 = 188.00
 188.00 - 0.25 = 187.75
 187.75 + 1.00 = 188.75
 188.75 - 0.25 = 188.50
 188.50 + 1.00 = 189.50
 189.50 - 0.25 = 189.25
 189.25 + 1.00 = 190.25
 190.25 - 0.25 = 190.00
 190.00 + 1.00 = 191.00
 191.00 - 0.25 = 190.75
 190.75 + 1.00 = 191.75
 191.75 - 0.25 = 191.50
 191.50 + 1.00 = 192.50
 192.50 - 0.25 = 192.25
 192.25 + 1.00 = 193.25
 193.25 - 0.25 = 193.00
 193.00 + 1.00 = 194.00
 194.00 - 0.25 = 193.75
 193.75 + 1.00 = 194.75
 194.75 - 0.25 = 194.50
 194.50 + 1.00 = 195.50
 195.50 - 0.25 = 195.25
 195.25 + 1.00 = 196.25
 196.25 - 0.25 = 196.00
 196.00 + 1.00 = 197.00
 197.00 - 0.25 = 196.75
 196.75 + 1.00 = 197.75
 197.75 - 0.25 = 197.50
 197.50 + 1.00 = 198.50
 198.50 - 0.25 = 198.25
 198.25 + 1.00 = 199.25
 199.25 - 0.25 = 199.00
 199.00 + 1.00 = 200.00
 200.00 - 0.25 = 199.75
 199.75 + 1.00 = 200.75
 200.75 - 0.25 = 200.50
 200.50 + 1.00 = 201.50
 201.50 - 0.25 = 201.25
 201.25 + 1.00 = 202.25
 202.25 - 0.25 = 202.00
 202.00 + 1.00 = 203.00
 203.00 - 0.25 = 202.75
 202.75 + 1.00 = 203.75
 203.75 - 0.25 = 203.50
 203.50 + 1.00 = 204.50
 204.50 - 0.25 = 204.25
 204.25 + 1.00 = 205.25
 205.25 - 0.25 = 205.00
 205.00 + 1.00 = 206.00
 206.00 - 0.25 = 205.75
 205.75 + 1.00 = 206.75
 206.75 - 0.25 = 206.50
 206.50 + 1.00 = 207.50
 207.50 - 0.25 = 207.25
 207.25 + 1.00 = 208.25
 20

$$\text{Rate} = \frac{\text{Cost of Service}}{\text{Revenue}} = \frac{10014.00}{10750.00} = 0.925$$

Period	Period	Wages
Aug. 1 - Oct. 1, 1912	Oct. 1 - Dec. 1, 1912	\$20.00
Nov. 1 - Dec. 1, 1912	Dec. 1 - Jan. 1, 1913	\$20.00

THE ECONOMIST

Longitude: $2^{\circ} 20' + 2^{\circ} 20' = 4^{\circ} 40'$ East

Tercer año

1872] *Schmid.*

2. $\lim_{n \rightarrow \infty} \left(\frac{1}{n} \right)^{\frac{1}{n}} = e^{-\frac{1}{n}}$

• C O M M U N I T Y •



Stringer's.

$$z \in \mathbb{H} \cup \mathbb{C}_+$$

The following table gives the results of the experiments.

$$100000 + 80000 = 180000$$

$\mathbf{F} = -\nabla \cdot \mathbf{B}$



Cost of Ledges & Pavement.

The cost of 18" wide strips of pavement is to be on the basis of \$1.50 per cubic yard. The cost of 12" wide strips of pavement is to be \$1.00 per cubic yard. The cost of 6" wide strips of pavement is to be \$0.60 per cubic yard. The cost of 3" wide strips of pavement is to be \$0.30 per cubic yard. The cost of 1" wide strips of pavement is to be \$0.15 per cubic yard. The cost of 1/2" wide strips of pavement is to be \$0.075 per cubic yard. The cost of 1/4" wide strips of pavement is to be \$0.0375 per cubic yard. The cost of 1/8" wide strips of pavement is to be \$0.01875 per cubic yard. The cost of 1/16" wide strips of pavement is to be \$0.009375 per cubic yard. The cost of 1/32" wide strips of pavement is to be \$0.0046875 per cubic yard. The cost of 1/64" wide strips of pavement is to be \$0.00234375 per cubic yard. The cost of 1/128" wide strips of pavement is to be \$0.001171875 per cubic yard. The cost of 1/256" wide strips of pavement is to be \$0.0005859375 per cubic yard. The cost of 1/512" wide strips of pavement is to be \$0.00029296875 per cubic yard. The cost of 1/1024" wide strips of pavement is to be \$0.000146484375 per cubic yard. The cost of 1/2048" wide strips of pavement is to be \$0.0000732421875 per cubic yard. The cost of 1/4096" wide strips of pavement is to be \$0.00003662109375 per cubic yard. The cost of 1/8192" wide strips of pavement is to be \$0.000018310546875 per cubic yard. The cost of 1/16384" wide strips of pavement is to be \$0.0000091552734375 per cubic yard. The cost of 1/32768" wide strips of pavement is to be \$0.00000457763671875 per cubic yard. The cost of 1/65536" wide strips of pavement is to be \$0.000002288818359375 per cubic yard. The cost of 1/131072" wide strips of pavement is to be \$0.0000011444091796875 per cubic yard. The cost of 1/262144" wide strips of pavement is to be \$0.00000057220458984375 per cubic yard. The cost of 1/524288" wide strips of pavement is to be \$0.000000286102294921875 per cubic yard. The cost of 1/1048576" wide strips of pavement is to be \$0.0000001430511474609375 per cubic yard. The cost of 1/2097152" wide strips of pavement is to be \$0.00000007152557373046875 per cubic yard. The cost of 1/4194304" wide strips of pavement is to be \$0.000000035762786865234375 per cubic yard. The cost of 1/8388608" wide strips of pavement is to be \$0.0000000178813934326171875 per cubic yard. The cost of 1/16777216" wide strips of pavement is to be \$0.00000000894069671630859375 per cubic yard. The cost of 1/33554432" wide strips of pavement is to be \$0.000000004470348358154296875 per cubic yard. The cost of 1/67108864" wide strips of pavement is to be \$0.0000000022351741790771484375 per cubic yard. The cost of 1/134217728" wide strips of pavement is to be \$0.00000000111758708953857421875 per cubic yard. The cost of 1/268435456" wide strips of pavement is to be \$0.000000000558793544769287109375 per cubic yard. The cost of 1/536870912" wide strips of pavement is to be \$0.0000000002793967723846435546875 per cubic yard. The cost of 1/107374184" wide strips of pavement is to be \$0.000000000139698386192321777296875 per cubic yard. The cost of 1/214748368" wide strips of pavement is to be \$0.00000000006984919309616088864375 per cubic yard. The cost of 1/429496736" wide strips of pavement is to be \$0.000000000034924596548080444321875 per cubic yard. The cost of 1/858993472" wide strips of pavement is to be \$0.0000000000174622982740402221609375 per cubic yard. The cost of 1/1717986944" wide strips of pavement is to be \$0.00000000000873114913702011108046875 per cubic yard. The cost of 1/3435973888" wide strips of pavement is to be \$0.000000000004365574568510055540234375 per cubic yard. The cost of 1/6871947776" wide strips of pavement is to be \$0.000000000002182787284255027770117296875 per cubic yard. The cost of 1/13743895552" wide strips of pavement is to be \$0.00000000000109139364212751388505859375 per cubic yard. The cost of 1/27487791104" wide strips of pavement is to be \$0.000000000000545696821063756942529296875 per cubic yard. The cost of 1/54975582208" wide strips of pavement is to be \$0.00000000000027284841053187847126459375 per cubic yard. The cost of 1/109951164416" wide strips of pavement is to be \$0.000000000000136424205265939235632296875 per cubic yard. The cost of 1/219902328832" wide strips of pavement is to be \$0.000000000000068212102632969617816146875 per cubic yard. The cost of 1/439804657664" wide strips of pavement is to be \$0.0000000000000341060513164848089080734375 per cubic yard. The cost of 1/879609315328" wide strips of pavement is to be \$0.0000000000000170530256582424044540367296875 per cubic yard. The cost of 1/1759218630656" wide strips of pavement is to be \$0.0000000000000085265128291212022272183459375 per cubic yard. The cost of 1/3518437261312" wide strips of pavement is to be \$0.00000000000000426325641456060111360917296875 per cubic yard. The cost of 1/7036874522624" wide strips of pavement is to be \$0.0000000000000021316282072803005568045859375 per cubic yard. The cost of 1/14073749045248" wide strips of pavement is to be \$0.00000000000000106581410364015027840229296875 per cubic yard. The cost of 1/28147498090496" wide strips of pavement is to be \$0.0000000000000005329070518200751392011459375 per cubic yard. The cost of 1/56294996180992" wide strips of pavement is to be \$0.00000000000000026645352591003756960057296875 per cubic yard. The cost of 1/112589992361984" wide strips of pavement is to be \$0.0000000000000001332267629550187848002867296875 per cubic yard. The cost of 1/225179984723968" wide strips of pavement is to be \$0.0000000000000000666133814775009392001433459375 per cubic yard. The cost of 1/450359969447936" wide strips of pavement is to be \$0.000000000000000033306690738750469600071667296875 per cubic yard. The cost of 1/900719938895872" wide strips of pavement is to be \$0.000000000000000016653345369375234800035833459375 per cubic yard. The cost of 1/1801439877791744" wide strips of pavement is to be \$0.00000000000000000832667268468761740001791667296875 per cubic yard. The cost of 1/3602879755583488" wide strips of pavement is to be \$0.00000000000000000416333634234380870000895833459375 per cubic yard. The cost of 1/7205759511166976" wide strips of pavement is to be \$0.000000000000000002081668171171904350004479167296875 per cubic yard. The cost of 1/1441151902233488" wide strips of pavement is to be \$0.000000000000000001040834085585452175002239583459375 per cubic yard. The cost of 1/2882303804466976" wide strips of pavement is to be \$0.0000000000000000005204170427927260875001179767296875 per cubic yard. The cost of 1/5764607608933952" wide strips of pavement is to be \$0.00000000000000000026020852139636304375000589883459375 per cubic yard. The cost of 1/1152921521786788" wide strips of pavement is to be \$0.00000000000000000013010426069818152187500029494167296875 per cubic yard. The cost of 1/2305843043573576" wide strips of pavement is to be \$0.000000000000000000065052130349090760937500014747083459375 per cubic yard. The cost of 1/4611686087147152" wide strips of pavement is to be \$0.000000000000000000032526065174545380468750000737354167296875 per cubic yard. The cost of 1/9223372174294304" wide strips of pavement is to be \$0.000000000000000000016263032587272690234375000036867723459375 per cubic yard. The cost of 1/1844674434858852" wide strips of pavement is to be \$0.0000000000000000000081315162936363451187500001843386167296875 per cubic yard. The cost of 1/3689348869717704" wide strips of pavement is to be \$0.00000000000000000000406575814681817255937500000921693083459375 per cubic yard. The cost of 1/7378697739435408" wide strips of pavement is to be \$0.0000000000000000000020328790734090862796875000004608465167296875 per cubic yard. The cost of 1/14757395478870816" wide strips of pavement is to be \$0.000000000000000000001016439536704543139375000002304232583459375 per cubic yard. The cost of 1/29514790957741632" wide strips of pavement is to be \$0.000000000000000000000508219768352271569687500000115211629167296875 per cubic yard. The cost of 1/59029581915483264" wide strips of pavement is to be \$0.000000000000000000000254109884176135784937500000057605814583459375 per cubic yard. The cost of 1/118059163830966528" wide strips of pavement is to be \$0.000000000000000000000127054942088067892468750000002880290729167296875 per cubic yard. The cost of 1/236118327661933056" wide strips of pavement is to be \$0.0000000000000000000000635274710440339462343750000001440145364583459375 per cubic yard. The cost of 1/472236655323866112" wide strips of pavement is to be \$0.000000000000000000000031763735522016973118750000000072007268229167296875 per cubic yard. The cost of 1/944473310647732224" wide strips of pavement is to be \$0.0000000000000000000000158818677610084865593750000000036003634114583459375 per cubic yard. The cost of 1/1888946621295464448" wide strips of pavement is to be \$0.000000000000000000000007940933880504243279687500000001800181705729167296875 per cubic yard. The cost of 1/3777893242590928896" wide strips of pavement is to be \$0.0000000000000000000000039704669402521213893750000000009000908528583459375 per cubic yard. The cost of 1/7555786485181857792" wide strips of pavement is to be \$0.000000000000000000000001985233470125560694687500000000450045426429167296875 per cubic yard. The cost of 1/1511157290036371584" wide strips of pavement is to be \$0.000000000000000000000000992616735051278037296875000000225022713214583459375 per cubic yard. The cost of 1/3022314580072743168" wide strips of pavement is to be \$0.00000000000000000000000049630836752563901868750000000011251135660729167296875 per cubic yard. The cost of 1/6044629160145486336" wide strips of pavement is to be \$0.00000000000000000000000024815418376281950937500000000056255678303459375 per cubic yard. The cost of 1/1208925832029097268" wide strips of pavement is to be \$0.0000000000000000000000001240770918814097546875000000002812783915167296875 per cubic yard. The cost of 1/2417851664058194536" wide strips of pavement is to be \$0.000000000000000000000000062038545940704937500000000001406391957583459375 per cubic yard. The cost of 1/4835703328116389072" wide strips of pavement is to be \$0.000000000000000000000000031019272970352487500000000000703195978783459375 per cubic yard. The cost of 1/9671406656232778144" wide strips of pavement is to be \$0.00000000000000000000000001550963648517624375000000000035159798939167296875 per cubic yard. The cost of 1/19342813312465556288" wide strips of pavement is to be \$0.000000000000000000000000007754818242588121875000000000175798994783459375 per cubic yard. The cost of 1/38685626624931112576" wide strips of pavement is to be \$0.00000000000000000000000000387740912129406093750000000008789949739167296875 per cubic yard. The cost of 1/77371253249862225152" wide strips of pavement is to be \$0.00000000000000000000000000193870456064703046875000000004394974869583459375 per cubic yard. The cost of 1/15474250649972445032" wide strips of pavement is to be \$0.00000000000000000000000000096935228032351521875000000002197487434783459375 per cubic yard. The cost of 1/30948501299944890064" wide strips of pavement is to be \$0.0000000000000000000000000004846761401617576093750000000109874371739167296875 per cubic yard. The cost of 1/61897002599889780128" wide strips of pavement is to be \$0.0000000000000000000000000002423380700808788046875000000054937185869167296875 per cubic yard. The cost of 1/123794005199779560256" wide strips of pavement is to be \$0.0000000000000000000000000001211690350404394409375000000027468592934583459375 per cubic yard. The cost of 1/247588010399559120512" wide strips of pavement is to be \$0.000000000000000000000000000060584517520219720468750000001373429646783459375 per cubic yard. The cost of 1/495176020799118240256" wide strips of pavement is to be \$0.0000000000000000000000000000302922587601098602187500000006867148123583459375 per cubic yard. The cost of 1/990352041598236480512" wide strips of pavement is to be \$0.0000000000000000000000000000151461293800549301093750000003433574061783459375 per cubic yard. The cost of 1/1980704083196472961024" wide strips of pavement is to be \$0.000000000000000000000000000007573064690027465054687500000017167870303583459375 per cubic yard. The cost of 1/3961408166392945922048" wide strips of pavement is to be \$0.000000000000000000000000000003786532345013732522343750000008583935150783459375 per cubic yard. The cost of 1/7922816332785891844096" wide strips of pavement is to be \$0.0000000000000000000000000000018932661725068662611875000000042919675751583459375 per cubic yard. The cost of 1/1584563266557788368816" wide strips of pavement is to be \$0.00000000000000000000000000000094663308625343313059375000000214598378753167296875 per cubic yard. The cost of 1/3169126533115576737632" wide strips of pavement is to be \$0.0000000000000000000000000000004733165431267165652968750000001072991593783459375 per cubic yard. The cost of 1/6338253066231153475264" wide strips of pavement is to be \$0.000000000000000000000000000000236658271563358282646875000000053649579689167296875 per cubic yard. The cost of 1/1267650613246230697552" wide strips of pavement is to be \$0.000000000000000000000000000000118329135781679141323437500000026824789844783459375 per cubic yard. The cost of 1/2535301226492461395104" wide strips of pavement is to be \$0.000000000000000000000000000000059164567890839570661875000000134123949223583459375 per cubic yard. The cost of 1/5070602452984922790208" wide strips of pavement is to be \$0.0000000000000000000000000000000295822839454197853309375000000067061974611783459375 per cubic yard. The cost of 1/10141204905969445580416" wide strips of pavement is to be \$0.000000000000000000000000000000014791141972709892765468750000003353098730583459375 per cubic yard. The cost of 1/20282409811938891160832" wide strips of pavement is to be \$0.00000000000000000000000000000000739557098635494638296875000000167654936529167296875 per cubic yard. The cost of 1/40564819623877782321664" wide strips of pavement is to be \$0.00000000000000000000000000000000369778549317747319146875000000083827468264583459375 per cubic yard. The cost of 1/81129639247755564643328" wide strips of pavement is to be \$0.000000000000000000000000000000001848892746588736595734375000000419137341329167296875 per cubic yard. The cost of 1/162259278495511129286656" wide strips of pavement is to be \$0.000000000000000000000000000000000924446373294368297867187500000209568670664583459375 per cubic yard. The cost of 1/324518556991022258573312" wide strips of pavement is to be \$0.000000000000000000000000000000000462223186647184148934375000001047843343329167296875 per cubic yard. The cost of 1/649037113982044517146624" wide strips of pavement is to be \$0.000000000000000000000000000000000231111593323592074467187500000523921671664583459375 per cubic yard. The cost of 1/129807422796408903429328" wide strips of pavement is to be \$0.000000000000000000000000000000000115555796661796037234375000002619608433329167296875 per cubic yard. The cost of 1/259614845592817806858656" wide strips of pavement is to be \$0.000000000000000000000000000000000057777898330898018617187500000130980421664583459375 per cubic yard. The cost of 1/519229691185635613717312" wide strips of pavement is to be \$0.00000000000000000000000000000000002888894916544900930854687500000654902108329167296875 per cubic yard. The cost of 1/1038459382371271227434624" wide strips of pavement is to be \$0.000000000000000000000000000000000014444474532722450465437500000327470451664583459375 per cubic yard. The cost of 1/2076918764742542454869248" wide strips of pavement is to be \$0.000000000000000000000000000000000007222237276361122232743750000163735225329167296875 per cubic yard. The cost of 1/4153837529485084909738496" wide strips of pavement is to be \$0.000000000000000000000000000000000003611118638180561113743750000081867612664583459375 per cubic yard. The cost of 1/8307675058970169819476992" wide strips of pavement is to be \$0.00000000000000000000000000000000000180555931909028055687187500000409338063329167296875 per cubic yard. The cost of 1/1661535011794033963895392" wide strips of pavement is to be \$0.000000000000000000000000000000000000902779659545140278437500000204669031664583459375 per cubic yard. The cost of 1/3323070023588067927790784" wide strips of pavement is to be \$0.00000000000000000000000000000000000045138982977257013921875000001023345158329167296875 per cubic yard. The cost of 1/6646140047176135855581568" wide strips of pavement is to be \$0.000000000000000000000000000000000000225694914886285069609375000005116725791667296875 per cubic yard. The cost of 1/1329228009435227171116336" wide strips of pavement is to be \$0.000000000000000000000000000000000000112847457443142534805468750000025583628958329167296875 per cubic yard. The cost of 1/2658456018870454342232672" wide strips of pavement is to be \$0.0000000000000000000000000000000000000564237287215712674029375000001279181447664583459375 per cubic yard. The cost of 1/5316912037740908684465344" wide strips of pavement is to be \$0.00000000000000000000000000000000000002821186436078563370146875000006395907238329167296875 per cubic yard. The cost of 1/1063382407548181736893068" wide strips of pavement is to be \$0.00000000000000000000000000000000000001410593218039281685074375000003197953619167296875 per cubic yard. The cost of 1/2126764815096363473786036" wide strips of pavement is to be \$0.000000000000000000000000000000000000007052966090196408425371875000001598976809583459375 per cubic yard. The cost of 1/4253529630192726947572072" wide strips of pavement is to be \$0.000000000000000000000000000000000000003526483045098204212718750000007994884048329167296875 per cubic yard. The cost of 1/8507059260385453895144144" wide strips of pavement is to be \$0.000000000000000000000000000000000000001763241522549102106871875000003997442024167296875 per cubic yard. The cost of 1/1701411852077090779028828" wide strips of pavement is to be \$0.00000000000000000000000000000000000000088162076117450505343750000001998721012083459375 per cubic yard. The cost of 1/3402823704154181558057656" wide strips of pavement is to be \$0.0000000000000000000000000000000000000004408103805872525271875000000099836050604167296875 per cubic yard. The cost of 1/6805647408308363116115312" wide strips of pavement is to be \$0.0000000000000000000000000000000000000002204051902936263135718750000049918025302083459375 per cubic yard. The cost of 1/1361129481661672623223064" wide strips of pavement is to be \$0.00000000000000000000000000000000000000011020259514681315678718750000024959012654083459375 per cubic yard. The cost of 1/2722258963323345246446032" wide strips of pavement is to be \$0.0055101297573406578393750000001247950632704167296875 per cubic yard. The cost of 1/5444517926646690

Bibliography.

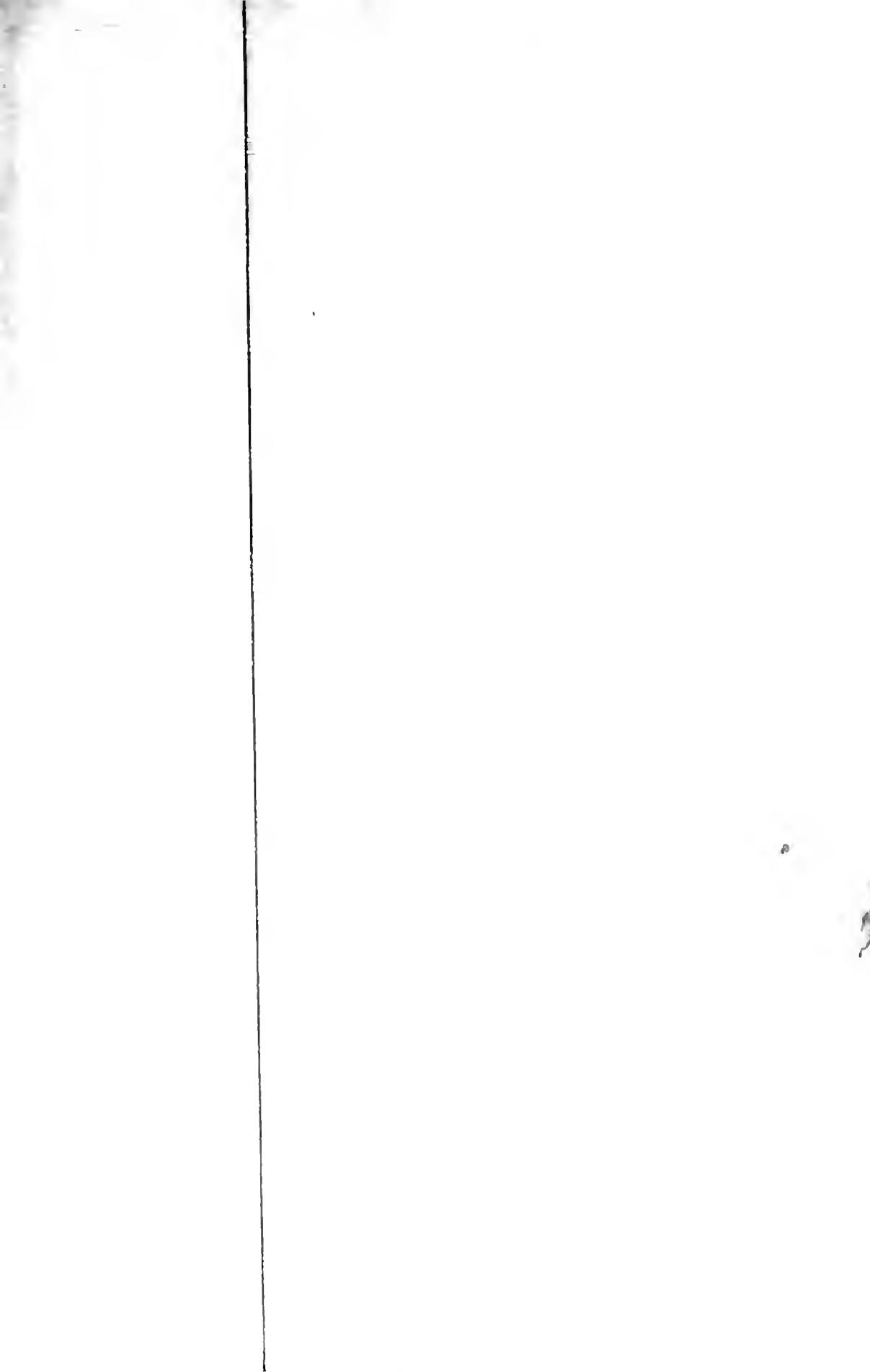
Wells. Steel Bridge Designing.

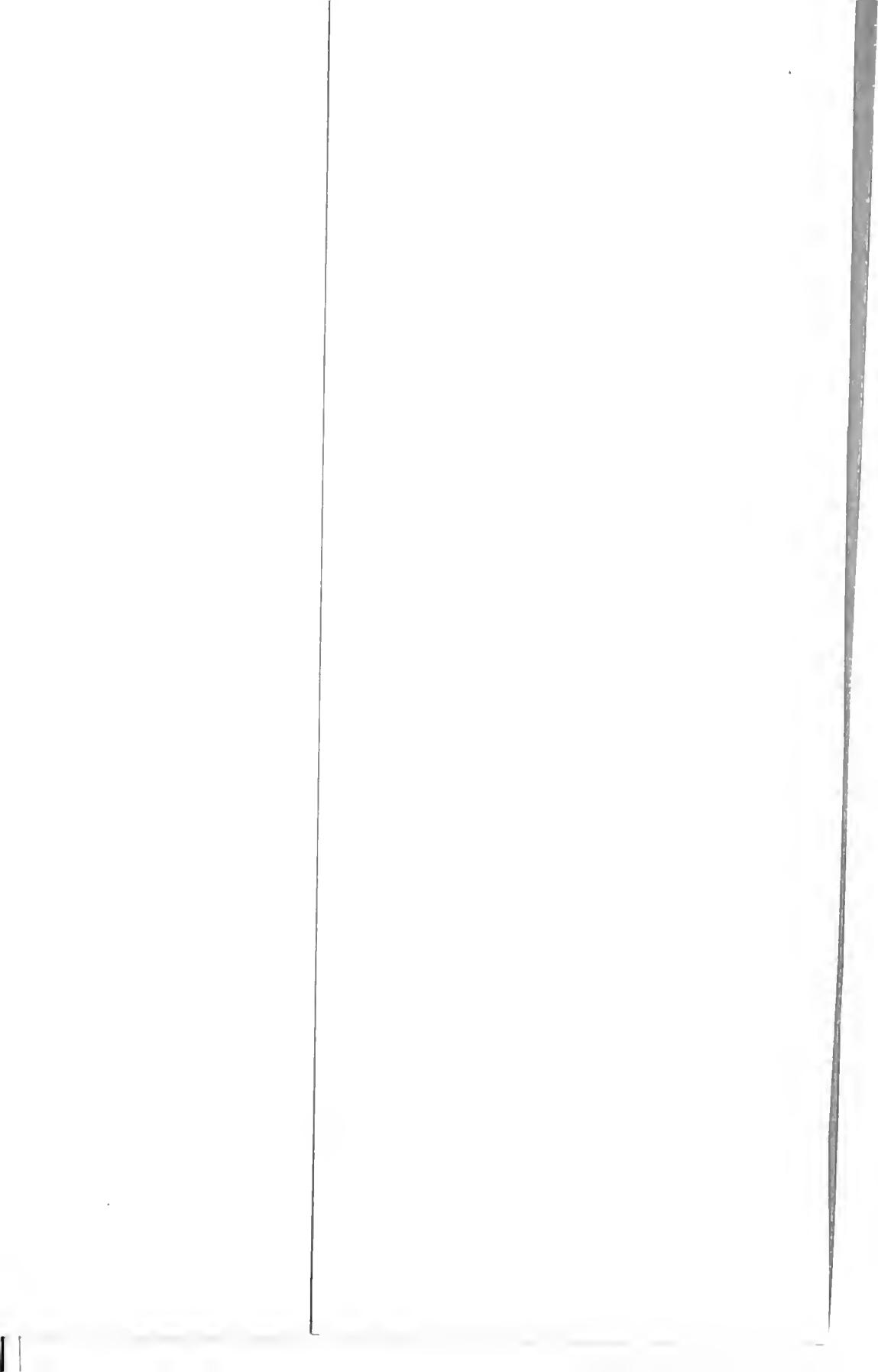
Turner and Lauver. Principles of Reinforced Concrete Construction.

Gillette. Cost Data.

Hand Book of Carnegie Steel Company.

Hand Book of Cambria Steel Company.





$$t = \pm \sqrt{c_4}$$

卷之三

— 1 —

12-
11-
10-
9-
8-

